PROCEEDINGS
THE INSTITUTION OF
CIVIL ENGINEERS

PART II
JUNE 1952 JUN 20 1952

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

15 January, 1952

Lieutenant-Colonel R. H. Edwards, M.I.C.E., Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Maritime Paper No. 19

"Oil Loading and Cargo Handling Facilities at Mina al-Ahmadi, Persian Gulf"

by

Charles William Niel McGowan, M.A., M.I.C.E., Reginald Charles Harvey, B.Sc.(Eng.), M.I.C.E., and James William Lowdon, B.Sc.(Eng.), M.I.C.E.

SYNOPSIS

What is believed to be the largest combined oil-loading and cargo-handling pier in the world has recently been completed at the north-west corner of the Persian Gulf, the purpose being to provide up-to-date arrangements for the rapid loading of tankers with crude oil.

The structure forms the principal item in a scheme which includes also a small-boat harbour for accommodating lighters and tugs and two buoy-type oil-loading berths for augmenting the delivery at peak demands.

The Paper covers the general principles of design involved and describes in detail several outstanding features, among which may be mentioned the facts that the pier is an all-welded open steel structure carried on steel piles, that gravity fendering has been incorporated, and that the whole of the steel was supplied and the erection undertaken by U.S. contractors to designs prepared by U.S. and British consultants in collaboration, and also in conjunction with the engineering department of the oil company.

The Paper includes a description of the oil loading, cargo handling, and other chief items of equipment.

A number of test piles were driven at the beginning of the work and the results of pulling some of these are given, as well as details of the driving of the large number of steel H-section piles. Reference is made to the marine life and adverse corrosion conditions which prevail in the Persian Gulf and particulars of the enamel and cathodic protection applied in order to protect the structure under water are included.

The welding of the flexible structure and the erection of the gravity fenders are described.

The erection of the pipework, which is an important part of the oil-loading arrangements as well as the cargo-handling and other equipment, is described and the plant and quantities of materials used are given. The Paper concludes with some notes on the working and maintenance of the piers since they were completed at the end of 1949.

Part I.—Planning and Design

by

C. W. N. McGowan, M.A., M.I.C.E.

INTRODUCTION

The year 1946 witnessed the start of the Kuwait Oil Company's crude oil production in the Sheikhdom of Kuwait at the north-west corner of the Persian Gulf (see *Fig. 1*). It was in June of that year that the first two pairs of submarine oil-loading lines, each consisting of two 12-inch-diameter lines, were brought into operation at a point on the Gulf coast then known as Fahaheel, and which has since been renamed Mina al-Ahmadi.

At that time the Company's base for the unloading of all stores was located at Shuwaikh on the southern shore of Kuwait Bay. The only means of landing cargo was by the use of a small barge jetty. Ships were able to approach no nearer than the anchorage 4 miles away and lighters brought the incoming cargo to the jetty, heavy lifts being handled by a 15-ton derrick at the head, whilst mobile cranes unloaded normal cargoes direct from the lighters on to trailers. A second small jetty to the westward was also available for occasional use.

By operating such limited facilities to maximum capacity, no less than 18,000 tons* were, on more than one occasion, brought ashore in one month, the average tonnage amounting to a figure not far short of this.

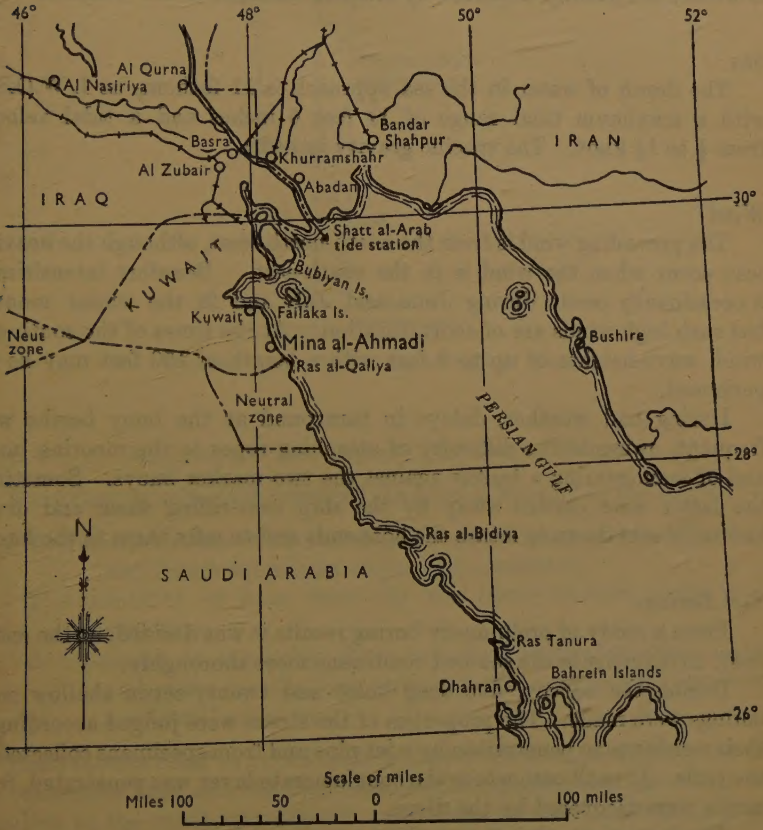
It was evident, however, that the Shuwaikh arrangements would have to be replaced as rapidly as possible by more adequate facilities. The Company therefore investigated the relative advantages of various available locations on the coast of the Persian Gulf and the final decision was taken in favour of Mina al-Ahmadi.

---

* Throughout the Paper "tons" refers to long tons of 2,240 lb. unless specified otherwise.

Among the advantages was the fact that Mina was only 5 miles from the Company's township of Ahmadi, whereas Shuwaikh was 24 miles away. It was also the Company's intention to continue to export all crude oil from Mina and, after commissioning the scheme, to discontinue the use of all but two pairs of the submarine lines which were becoming incapable of loading the tankers at the required rate. These comprised five buoy-berths about 4,000 feet from the shore with two 12-inch submarine lines to each mooring from the shore manifold. Only the northern and southern moorings A and E now remain in service, the three intermediate ones having been removed at intervals during the course of construction.

Fig. 1



LOCATION MAP

Site Conditions

Shore

The beach and foreshore are mostly sand with scattered areas of soft sandstone. The slope of the beach is gradual at the site, the gradient being approximately 1 in 66. There has been no indication to date of any large-scale sand movement, although accumulations do occur; these, however, are usually dispersed by seasonal changes in the weather.

Sea

The depth of water in the sea approach is 11 fathoms at L.W.O.S.T. with a maximum tidal range of 11 feet 6 inches and a tidal velocity from $\frac{1}{2}$ to $1\frac{1}{2}$ knot. The specific gravity is 1·029.

Wind

The prevailing wind is from the north-north-west, although the heaviest seas occur when the wind is in the south-east. Beaufort intensities of 8 occasionally occur during June and July and in the winter months, but such high winds are of short duration. At the times of the south-east wind, wave-heights of up to 8 feet with a length of 150 feet may be experienced.

During bad weather, delays in turnround at the buoy berths were frequent, owing to the difficulty of attaching ropes to the mooring buoys and of manoeuvring a tanker against the two marker buoys. Sometimes the latter were carried away by the ship over-riding them and divers had to be sent down to locate the hose ends and to refix them to the buoys.

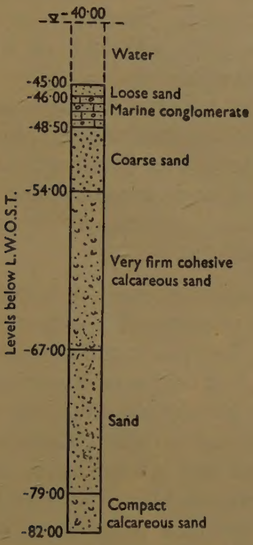
Test Borings

From a study of preliminary boring results it was decided, at the end of 1947, to investigate the sea-bed conditions more thoroughly.

During the survey, five deep holes and twenty-seven shallow wash borings were made. The properties of the strata were judged according to their resistance to penetration by a jet pipe and from specimens collected by the tools. In each case where the conglomerate layer was penetrated, fragments were recovered by the diver.

The survey revealed that a thin crust covered the sea bed over much of the site. This stratum was in no case found to exceed 3 feet 3 inches in thickness and open cracks indicated the possibility of large slabs beneath a thin covering of sand. The composition of the conglomerate was shells and sand, cemented by a hardened mud. The strata beneath were found to be generally of a soft sandy nature with scattered calcareous beds. Typical results are given in *Fig. 2*.

Fig. 2



TYPICAL TEST BOREHOLE AT CARGO PIER

OIL-LOADING AND CARGO-DISCHARGING PIERS

The following criteria determined the precise location of the scheme :—

1. Non-interference with the shore loading arrangements of present and future submarine oil-loading operations.
2. Availability of deep water for the larger tankers under construction.
3. Proximity to a suitable shore area for the establishment of oil and power installations, storage yards, maintenance facilities, residential areas, etc.

These factors ruled out the foreshore part of the area occupied by existing submarine lines and the area between the manifold and the village of Shiaibah to the south was selected on account of the firmer and higher ground, these being more suitable for the shore establishments.

Capacity Requirements

Jetty-, dolphin-, and buoy-types of oil berths suitable for open sea were considered. The number of days on which it would not be possible for a ship to come alongside because of inclement weather was of paramount importance to prevent "bunching" of tankers, and weather records showed that jetty berths might be out of action for 15–20 days in a year, dolphins 30–40 days, and buoy moorings 50–60 days. It was probable also that for considerable periods each year the lighterage method of cargo discharge would be interrupted. Similar interruption might indeed arise for cargo unloading at a jetty, although oil loading under those same conditions might remain possible.

Again, there was the question of the time actually involved in turn-round of ships and it was determined that dolphins would have an advantage over moorings and a jetty over dolphins. It was therefore concluded that a combined oil-loading and cargo-discharging pier would most satisfactorily meet all conditions. Accordingly designs were initially prepared for the following requirements :—

(*a*) Daily average export of crude oil: 500,000 barrels.*

(*b*) Monthly average import of dry cargo: 19,000 tons.

(*c*) Accommodation for the simultaneous loading of six tankers consisting of four berths on the outside and two on the inside.

(*d*) Accommodation for the simultaneous discharge of two general cargo vessels, one on the outside and one inside, with one berth provided with a fixed heavy-lift derrick.

(*e*) 40 feet depth of water at low tide at all berths.

(*f*) Construction programme to be based on a policy of completion and operating in the minimum possible time.

In view of the gradual fall of the sea bed from the shore and the manœuvring space for tankers using the inside berths, it was necessary to locate the head of the pier no less than 4,200 feet into the Gulf, access to be by way of an approach pier with a pipeway alongside.

Selection of Types of Piles

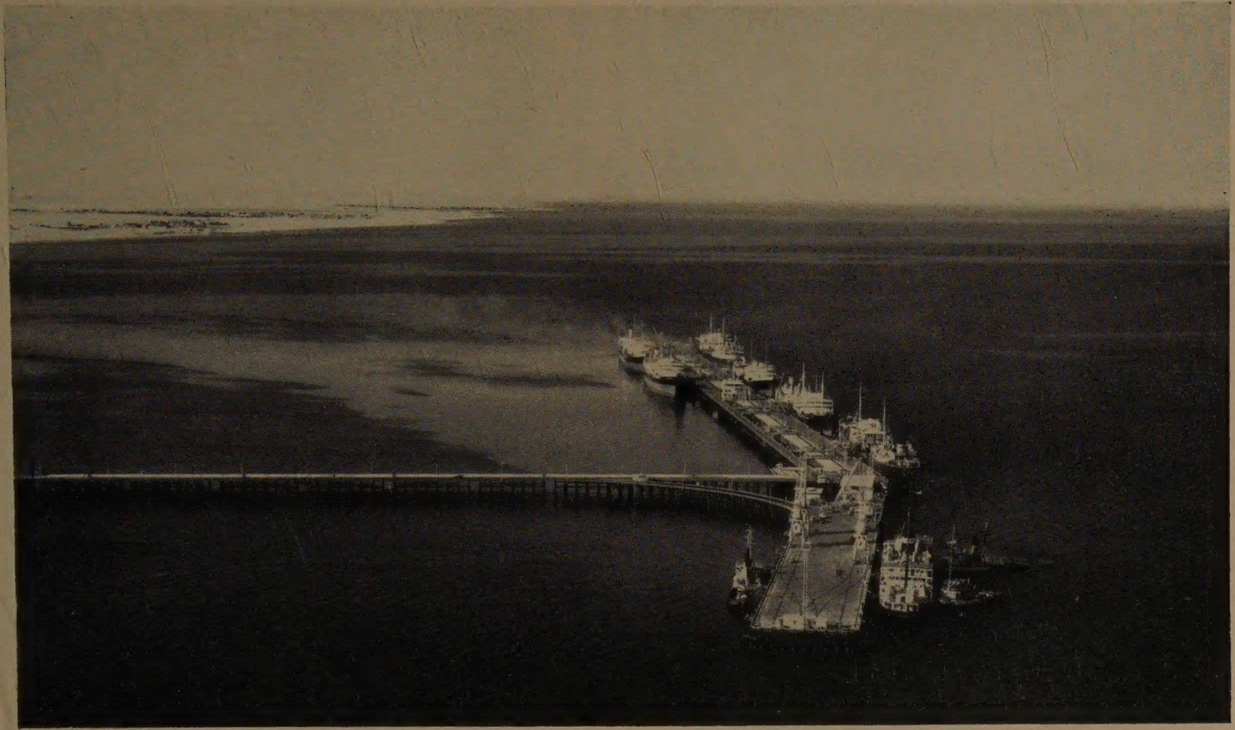
Pre-cast concrete, creosoted timber, and H-section steel piles were each carefully considered in turn. For pre-cast piles to be satisfactory two conditions would have to be satisfied: first, an adequate supply of local aggregate suitable for producing a concrete sufficiently dense to withstand driving and also continuous exposure in Persian Gulf water; and secondly, a sea bed which would allow sufficient penetration of the piles. Neither of these conditions was obtainable and since maximum speed of construction was essential, the risk of loss of time and money turned the decisions against this form of pile.

Creosoted timber presented a long and costly haulage problem, an increased fire hazard, and the necessity for driving many more such piles than would be the case with either reinforced-concrete or steel piles. Furthermore, the risk of attack by marine borers is ever present, especially in water the temperature of which rises during the summer to 90° F. It was therefore decided to adopt an H-section steel pile which had the advantage that any welding to the required lengths could be accomplished at the site. Furthermore, steel piles, by reason of their flexibility,

---

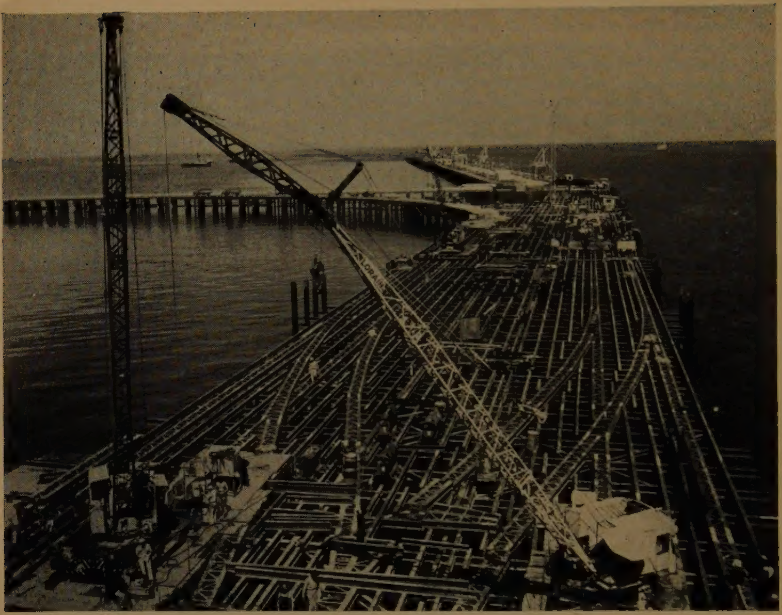
* 7·4 barrels = 1 ton; 1 barrel = 35 imperial gallons.

Fig. 4



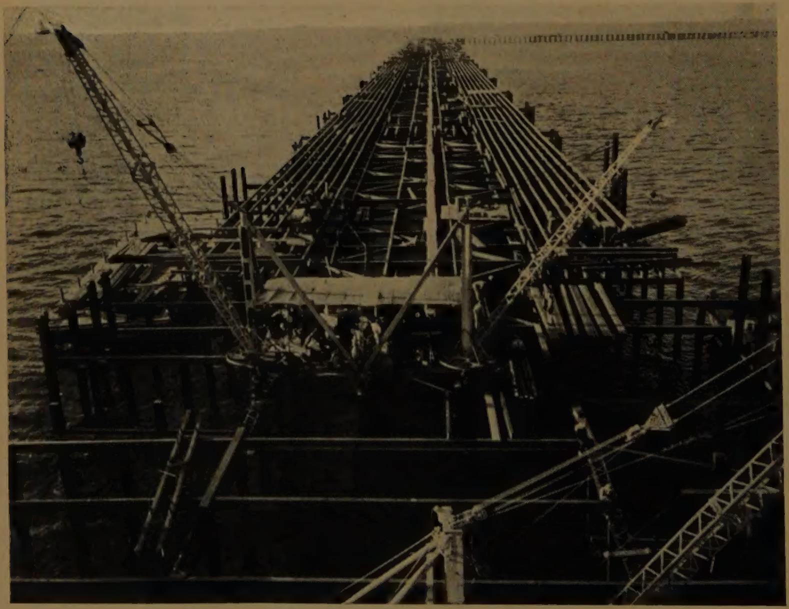
General Aerial View

Fig. 10



Cargo Pier—General View of Deck Steelwork

Fig. 17



Oil Pier—General View of Deck Steelwork

are able to withstand the impact from ships more satisfactorily than reinforced-concrete piles, because of better absorption of the blow.

It had, however, to be recognized that corrosion would present a big problem, and this is discussed in detail in the second part of the Paper.

Source of Supply of Steelwork

After exhaustive enquiries regarding the possibilities of obtaining the steelwork in either the United Kingdom or the United States of America it was found to be impossible to locate suppliers in the United Kingdom who could guarantee delivery within the time specified. As a result, the decision was taken to employ a firm of American constructional engineers to supply and fabricate the whole of the material in the structure and also for the complete erection.

It so happened that the firm approached was engaged at the time on major works elsewhere in the Persian Gulf. The firm in question, through their associated U.S. consultants, was prepared to furnish the complete scheme in detail from proposals set out beforehand by the British Consultants to the Oil Company, also to work in close co-operation with steel works in the U.S.A. in the production of the rolled sections.

Specifications

Details were prepared in accordance with the following :—

1. American Institute of Steel Construction Specifications, 1947, for the Design, Fabrication and Erection of Structural Steel for Building.
2. American Welding Society Standards.
3. Recommendations of the National Lumber Manufacturers' Association.

Where the A.I.S.C. Specifications were not applicable, the Standard Specifications for Highway Bridges in the U.S.A. were followed.

The designed stress for the steel members under normal loading is 20,000 lb. per square inch, with the standard reduction factor embodied so far as columns and beams are concerned. Where berthing impact is involved a higher figure of 29,000 lb. per square inch has been assumed.

Wind load allowances are :—

(*a*) Structure : 20 lb. per square foot on a vertical plane normal to the assumed wind direction.
(*b*) Live load : 200 lb. per linear foot.

Lateral load allowances provide for :—

(*a*) Oil and Cargo Piers : 400,000 lb. per ship due to wind, waves, and surge on the ship when lying alongside.
(*b*) Approach Pier : 300 lb. per linear foot of structure.

Throughout all the calculations a temperature range of ± 50° from the mean temperature of 80° Fahrenheit has been assumed.

GENERAL STRUCTURAL DESIGN

The pier is an all-welded steel structure with timber decking and consists of three main units, namely, the Approach Pier, the Oil Pier, and the Cargo Pier, together forming an unsymmetrical T-shape in plan as shown by Fig. 3, Plate 1, and *Fig. 4*. The approach pier carries a 24-foot-wide roadway, 4,140 feet long, and extends eastward from the shore with a gradient rising from a level of + 16·00 (feet) above low-water at the shore end to + 23·00 at the eastern extremity, at which point connexion is made with the oil-loading arm extending to the north, and with the cargo arm to the south, the latter by a curved approach at a radius of 333 feet 9 inches. Adjacent to the north side of the roadway is a 35-foot-6-inch pipeway supporting eight steel 24-inch-diameter crude-oil lines, as well as steel service lines, comprising fresh water, salt water (for fire), and bunker and diesel fuel (see Fig. 7, Plate 1). Space has been left for a future 20-inch line.

The oil pier is 2,805 feet long by 105 feet wide, whilst the cargo arm is 870 feet long by 100 feet wide. Figs 5 and 6, Plate 1, illustrate the general lay-outs of both.

STRUCTURAL DESIGN OF APPROACH PIER

General Description

It will be seen from Figs 11, Plate 2, that the approach pier comprises a series of bearing piles, continuous cap beams, diagonal bracings, and horizontal struts, all above low-water level and each bent forming a portal frame of several panels. In case difficulties arose which might prohibit the use of floating plant, an interval of 15 feet between bents was adopted which would enable a skid-type pile driver to be employed.

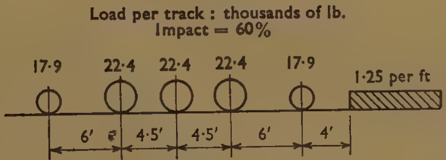
In designing the bents as portal structures, the piles were assumed to be " fixed " 5 feet below the mud-level. This assumption agrees reasonably well with the result of an actual deflexion test which is referred to in Part II.

The piles employed throughout are U.S. 14BP73—14 inches by 14½ inches by 73 lb. per foot, with a web and flange thickness of 0·506 inch.

The deck loading assumed for the design of the approach pier makes provision for two rail tracks (see *Fig. 12*), for moving loads comprising two 13·5-ton lorries passing, a 22·5-ton mobile crane, and a uniformly distributed load of 200 lb. per square foot. From the junction with the curved cargo-pier approach on the length connecting with the oil pier together with the whole of the oil-pier deck, the loading has been reduced to two 4·5-ton lorries passing, thus enabling the thickness of the timber deck to be reduced accordingly. From the shore end to the junction referred to, the decking consists of laminations of 6-inch-by-3-inch West Coast Douglas Fir. No handrails have been fitted but protection at the edges comprises low timber curbs as wheel guards. All timber for the decking has been pressure-treated with " Tanalith " (Wolman Salts) as a preservative, which also acts as a fire retardant. The effectiveness of this treatment has been amply demonstrated by tests on the surface-spread of flame, carried out by the Fire Research Station, Boreham Wood, Hertfordshire.

It will be observed that two pairs of rail bearers have been incorporated; they are designed to carry a metre-gauge railway with loads as shown in *Fig. 12.* A later decision was taken to omit the railway, but provision has been made, by retaining the bearers, for the possible installation of the track at some future date.

Fig. 12



RAIL TRACK LOADING

Rail-stringer stresses were computed by including traction force or braking force with the wheel loads plus impact.

Pipeway

The pipeway was designed as an integral part of the approach pier by considering the structure between the expansion joints as a series of units. These units comprise eight bents and are each 120 feet long, except for the units at the shore and eastern ends. Alternate bents have been extended northwards to provide support at 30-foot spacing for the pipelines, all pipes being mounted on rollers fitted with Monel-metal shafts to minimize resistance to movement. With the use of such rollers the frictional forces resulting from the pipeline movement amount to only 7·5 per cent of the vertical loading. To take the forces resulting from the unequal expansion and contraction of the pipelines, longitudinal bracing is incorporated in each 120-foot unit throughout the pipeway.

Adjacent to the rock-fill causeway the pipeway is constructed of two-bearing-pile bents with continuous cap beams cantilevering from each pile.

Expansion Joints

Joints are provided between units which permit movement longitudinally, the gap being ½ inch at 80° F., and the timber decking stopping at angles on each side of the joint.

At the junction of the approach pier with the oil pier is fitted a deflexion joint designed to allow for movement of the oil pier, both along its longitudinal axis and at right angles to it.

To allow for longitudinal temperature-movement there is a complete gap, the separation being limited to 1 inch by two flexible tension-members made up of 2-inch-by-$\frac{5}{16}$-inch bars, 14 feet 8 inches long. These are tied to adjacent sides of the gap and are free to buckle as the gap closes.

Lateral movement of the oil pier caused by berthing impact and bollard pull is provided for by allowing the stringers of the last approach-pier bay to slide in and out between those of the next bent. Continuity at deck-level is provided by a sliding-finger joint.

The range of lateral movement allowed by the deflexion joint is as follows :—

	Outward (bollard pull)	Inward (berthing impact)
Designed:	4 inches	9 inches (+ 1-inch gap)
Anticipated:	2½ ,,	6½ ,,

This deflexion joint is illustrated in Figs 13 (*a*), Plate 2.

The curved portion of the approach pier was designed to be similar to the straight portion, expansion joints being incorporated in a like manner. There is also a deflexion joint at the junction of the curved portion and the cargo pier which permits a movement of 7 inches.

STRUCTURAL DESIGN OF OIL PIER

General Description

The oil pier consists of eight-pile bents with the deck stiffened along the centre-line at cap-beam level by a continuous horizontal truss.

The bents are made up of two "portal frames" (see Fig. 8, Plate 2, and Figs 14, Plate 3) connected by a continuous pile cap-beam with the addition of simple struts at the lower levels, the spacing of the bents again being 15 feet.

Horizontal reactions resulting from berthing are absorbed first by the four fender piles in a group, and then by three fender bents, and finally by the horizontal truss which can be seen in *Fig. 17* (facing p. 255).

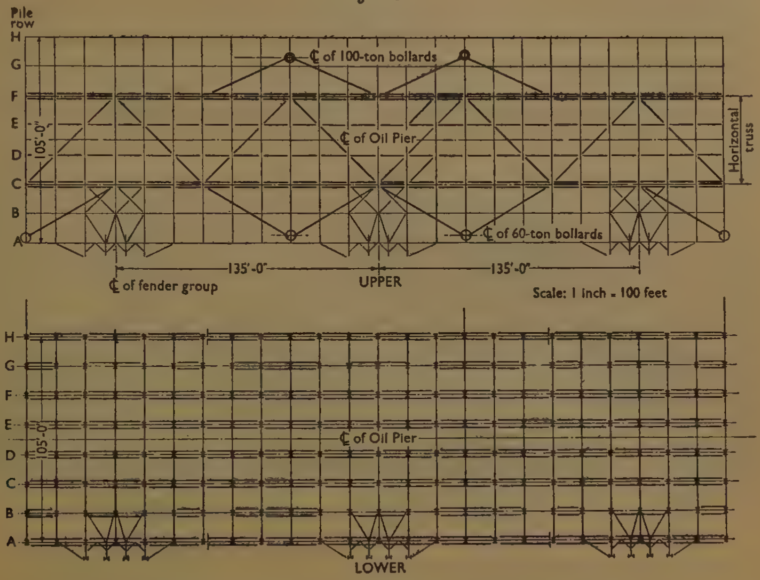
The decking is laminated Douglas Fir, of 4-inch-by-2-inch section, and is treated in a similar manner to that on the approach pier and cargo pier.

Horizontal Truss

The function of this most important feature in the design, besides being to distribute the horizontal reactions due to berthing, is to distribute the pulls on the bollards and the forces caused by pipeline movements.

The truss is 45 feet deep on the oil pier (and 20 feet on the cargo pier) between the centre-lines of chords, these being made of twin laced channels placed back to back, and separated by a distance equal to the depth of the pile. The channels vary in size from 12-inch-by-20·7 lb. to 15-inch-by-40 lb.

Figs 15



Oil Pier—Upper and Lower Horizontal Bracing

The system of diagonals is so arranged that panel points occur opposite the centres of fender groups (Figs 24, Plate 4) so that berthing loads are correctly transmitted to the truss. Diagonals are composed of wide-flanged beams of varying sizes.

In order that the truss should operate satisfactorily no expansion joint could be introduced, and there is a possible total expansion of 11 inches. The pile tops at each end of the pier with this increase would therefore move about 5½ inches from the mean position and at this deflexion the maximum bending stress in the piles will be approximately 11,600 lb. per square inch—a figure not considered excessive having regard to their flexibility. Also, to enable the truss to take up the lateral loads more freely, cuts have been introduced at intervals in the stringers along the outer pile rows A and H. These cuts are seen in Fig. 13 (*b*), Plate 2.

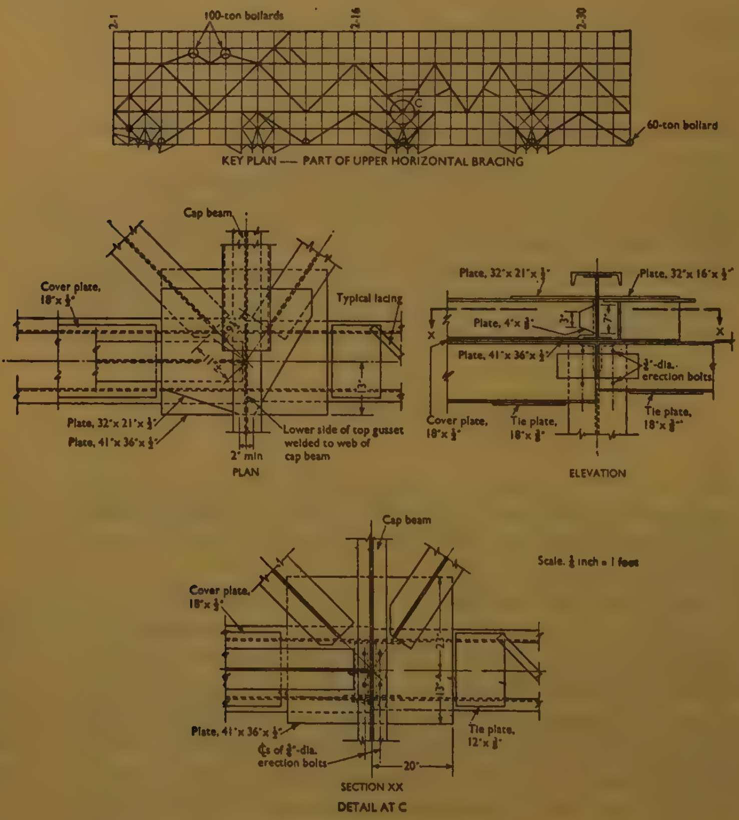
Additional stiffness has been provided by adding auxiliary chord members at each panel point and also transverse trusses at bents 2–47 and 2–124 (the points at which the 12-inch, 16-inch, and 24-inch pipes are attached to the pier by spring-loaded anchorages), at bents 2–18 and 2–166 where the lateral forces from the pipes occur, also at bent 2–82, this being the position of the vertical expansion loop to which reference is made later in the Paper.

Pipeway

The pipeway (*Fig. 18*) lies on the centre-line with a 30-foot roadway on the east and a 15-foot or 30-foot road on the west side, connected every 100–150 feet by 30-foot-wide bridges.

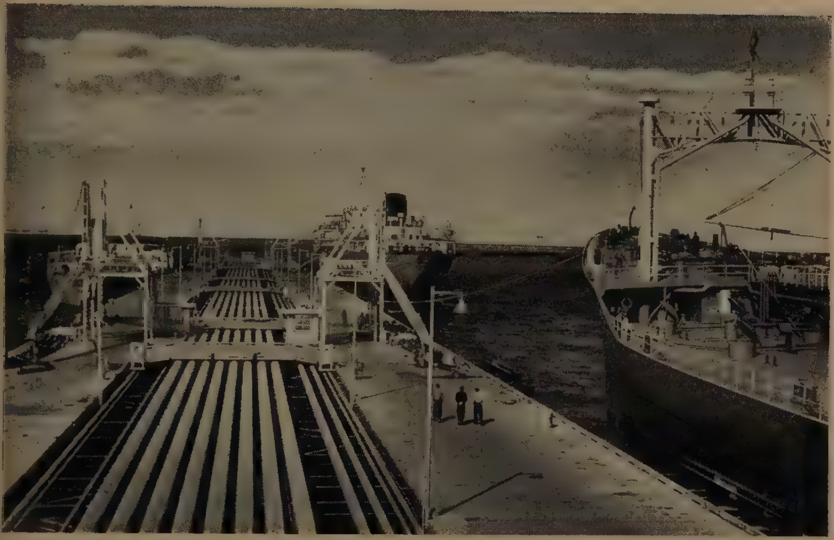
At the junction of the approach pier and oil pier a Z-shaped expansion bend with 85-foot and 92-foot legs is introduced into each of the eight 24-inch-diameter lines to provide for a possible movement amounting to 22½-inches in the approach-pier lines and 2½ inches in the oil-pier lines. The roller supports carrying the Z-bends (*Fig. 19*) allow for both longitudinal and lateral movement.

Figs 16



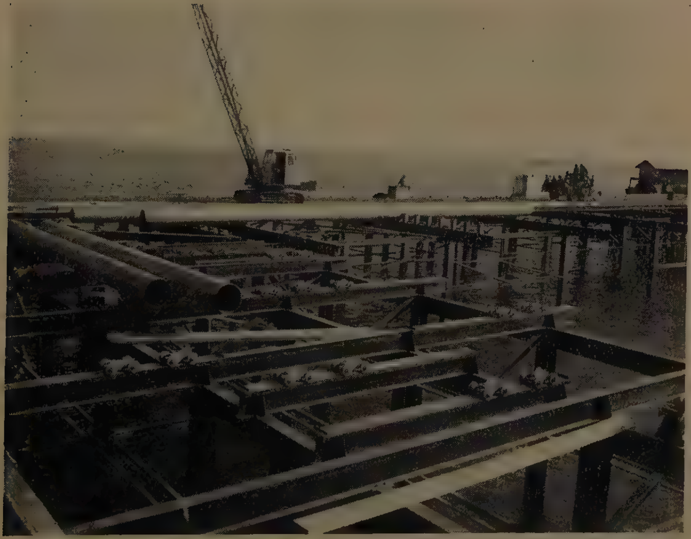
Oil Pier—Typical Horizontal Truss Connexion

Fig. 18



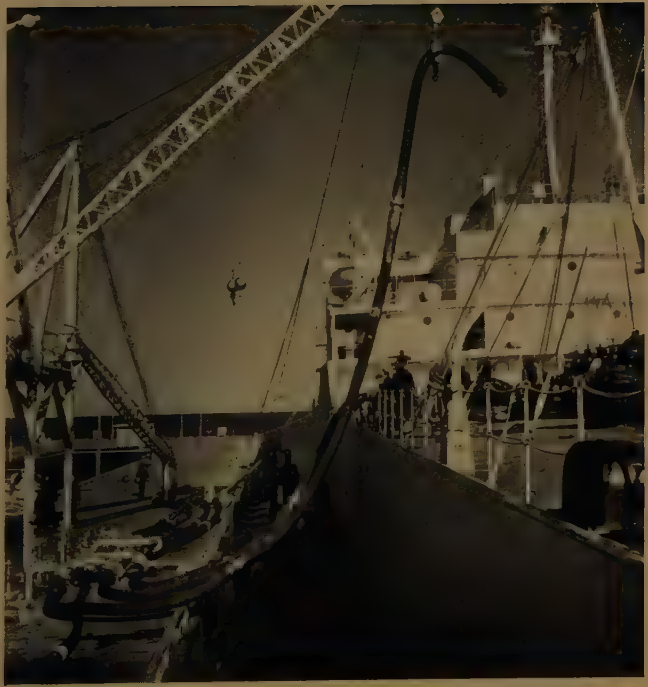
Oil Pier, showing Berths Nos III, IV, V, and VI

Fig. 19



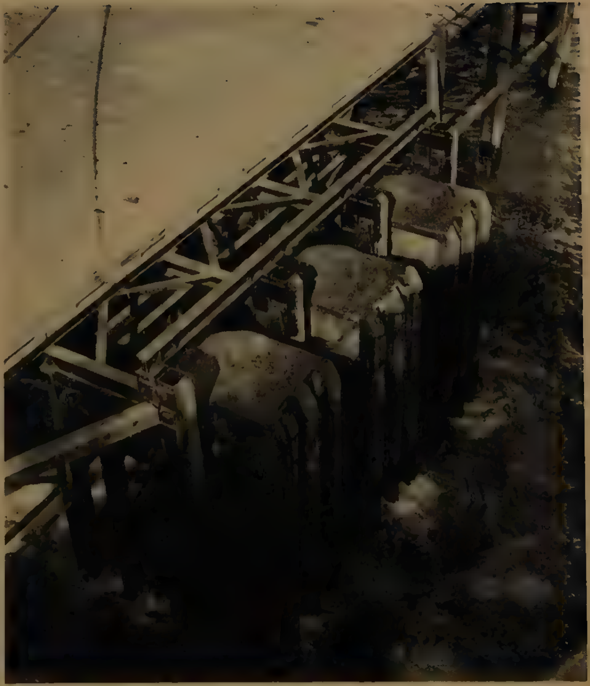
Universal Rolls at Z-Bends

Fig. 20



Oil Delivery Hoses

Fig. 25



Oil Pier—A Group of Gravity Fenders

Structural Design of Cargo Pier

General Description

The design of the cargo pier is, in general, similar to that of the oil pier, the deck structure being carried on piles spaced at 10-foot centres in both directions for carrying the heavier uniform deck load of 600 lb. per square foot. A typical cross-section is shown in Fig. 9, Plate 2.

The cargo pier is separated from the oil pier by a decked-over cross-connexion, 170 feet long by 60 feet wide. Deflexion and expansion joints, as previously mentioned, have been provided at the junction with the approach pier, and also a complete gap at the junction of the cross-connexion and oil pier. The gap is bridged at deck-level by a $\frac{3}{4}$-inch-thick steel plate extending the width of the deck and permitting a movement range of 15 inches. Figs 13 (*c*), Plate 2 show the expansion joint on the outer pile-rows.

The deck is of 6-inch laminated timber throughout, and treated as for the other decking previously described. Provision has also been made for carrying the four railway tracks if required.

Deck and Deck Bearers

To guard against lateral buckling of the stringers under the heavy deck loads to be provided for, diaphragms have been inserted between them at selected points. Stiffeners have been embodied at all bearing points between the cap beams and stringers and in the cap beams themselves where the stringers are cut.

Additional piles for carrying the stiffleg derrick, referred to later, were provided, capped by steel grillages to support the mast and each leg, whilst the hoist base rests on beams carried by the deck stringers.

Valve-Operating Platforms

Platforms at the service points are situated 9 feet below the deck, access being by steel ladders placed outboard in order to avoid cutting openings in the deck proper. A steel grating above the plated floor (see Figs 29, Plate 4) allows spilled oil to drain to a sump from which it is returned by a compressed-air-driven pump to the 6-inch salvage line.

Oil- and Cargo-Pier Fenders and Bollards

Fenders

The gravity-type fender, which was employed on a jetty[1] for the first time at Heysham, Lancashire, was selected as the most appropriate for meeting the problem of open-sea berthing of large vessels. The device as applied to the Mina piers is illustrated by Figs 21 and 22, Plate 3, Figs 23 and 24, Plate 4, and *Fig. 25*. These show cylinders, 21 feet 3 inches long by

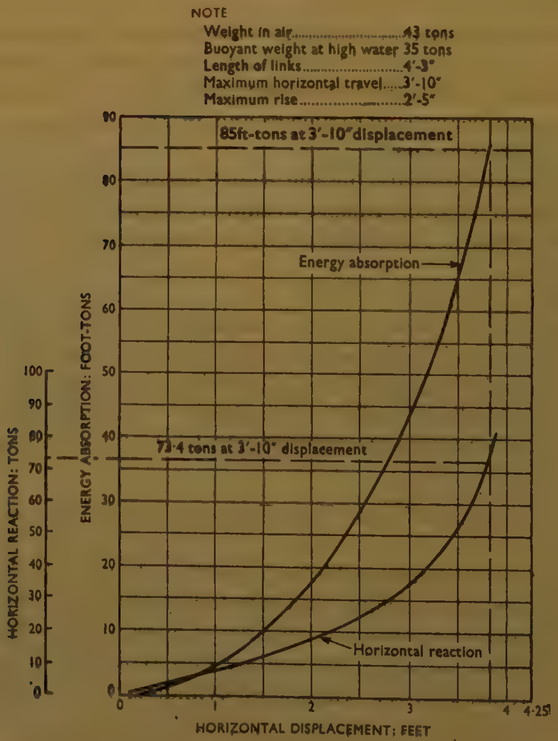
---

[1] Prof. A. L. L. Baker, "The Heysham Jetty," Maritime Paper No. 9, Instn Civ. Engrs, 1948.

6 feet diameter, formed of $\frac{5}{16}$-inch-thick steel plate, and filled with concrete, the filling operation having been carried out after the cylinders had been placed in position. The weight, in air, of each is 43 tons and correspondingly less when immersed. Figs 21, Plate 3, show the upper and lower ends fitted with 6-inch-diameter stainless-steel trunnion pins through the cylinders; corresponding pins are attached to the structure. Four forged suspension links (two at the upper end and two at the lower end) carry each cylinder, the lengths of each set of four being matched so that the distance between bearing surfaces shall not vary more than $\frac{1}{32}$ inch. The links are forged out of $2\frac{1}{2}$-inch-diameter anchor-chain steel and tested to withstand a pull of 300,000 lb. without permanent distortion.

The cylinders are arranged in groups of three at 10-foot centres, as shown in Figs 21, Plate 3, the groups being carried by four-pile bents parallel to the longitudinal axis of the pier and located 8 feet from the outer row of piles. At each oil berth the groups are symmetrically located at 60 feet and 195 feet on either side of the centre of the oil-loading

Fig. 26


NOTE
Weight in air........................43 tons
Buoyant weight at high water 35 tons
Length of links.......................4′-3″
Maximum horizontal travel.....3′-10″
Maximum rise.........................2′-5″

Characteristic Curves for Single Oil-Pier Fender at High Water points. The remaining groups are spaced to accord with the varying berth lengths.

In the design the effective average fixed-end length of the fender piles is taken from 5 feet below the sea bed and up to a point one-third of the distance between planes of lateral bracing measured upwards from the lowermost bracing. For the average bed-level of —45·00 and bracing planes at +12·00 and +20·00 the effective fixed-end pile-length becomes 64·67 feet or 776 inches. The design limits the horizontal force transmitted by any one fender group subject to the designed maximum berthing load to 500,000 lb.

The maximum impact load has been taken as the result of a 30,000 ton (dead-weight)* tanker at the oil pier and a 24,000 ton cargo ship at the cargo pier, striking a fender group at 0·75 knots, assuming 40 per cent of the energy to be absorbed by the fender group and the pier and the balance by deformation of the ship's hull. In the case of the tanker the energy to be absorbed by the fender group and pier is found to be 672,000 foot-lb. Characteristic curves for a single oil-pier fender are shown in *Fig. 26*. Vertical 10-inch-by-10-inch rubbing strips, bolted to the outer pile rows, give protection to the piers at all other points.

Bollards

The spring and head-line bollards are arranged at 60, 150, and 285 feet on each side of the oil-loading points and 4 feet from the edge of the deck. The positions of the various bollards are indicated in Fig. 5, Plate 1. The breast-line bollards are designed to withstand a 100-ton pull through an angle of 60 degrees, the spring and head-line bollards a pull of 60 tons through 180 degrees, and the tug bollards on berths VII and VIII only, a pull of 15 tons through 180 degrees.

The cargo-pier front bollards are spaced at about 80-foot centres on both sides, whilst four back bollards are also provided, all being designed to take a pull of 40 tons through an angle of 180 degrees.

Each bollard comprises a concrete-filled steel pipe, welded to a cap beam and to a longitudinal with large corner gussets.

OIL LOADING FACILITIES

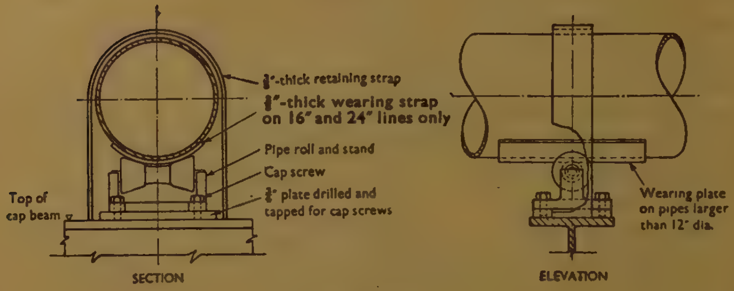
A concrete anchorage for all the loading lines has been provided by means of a block, 30 feet by 36 feet by 8 feet thick, weighing 660 tons, located at the shore end of the approach. On reaching the eastern end the eight 24-inch lines are spaced by special pipe-guides before they turn through the Z-bends on to the centre-line of the oil pier along which they extend about 2,325 feet to the terminal manifold. *Figs 27* illustrate these guides.

All mains are fitted with valves at the terminal manifold so that any line can be connected with any other.

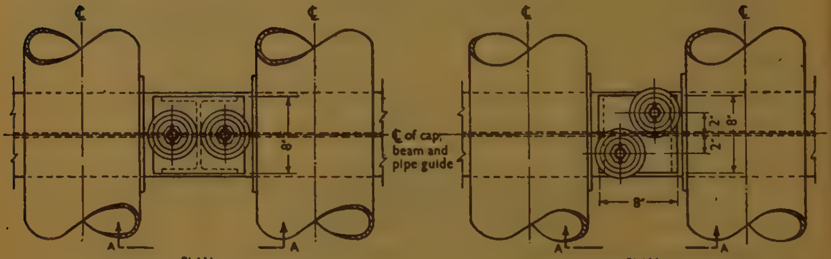
---

* *For* "dead-weight" *read* "displacement." (See discussion by Mr J. A. Williams, p. 312.

Figs 27

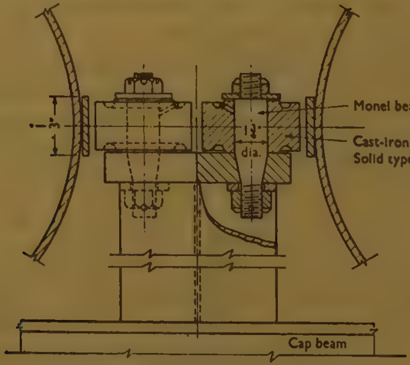

⅜"-thick retaining strap
⅜"-thick wearing strap on 16" and 24" lines only
Pipe roll and stand
Cap screw
¾" plate drilled and tapped for cap screws
Top of cap beam
SECTION
Wearing plate on pipes larger than 12" dia.
ELEVATION

TYPICAL PIPE ROLL



PLAN
(for clearance = 12" or more)

PLAN
(for clearance = 10" or less)

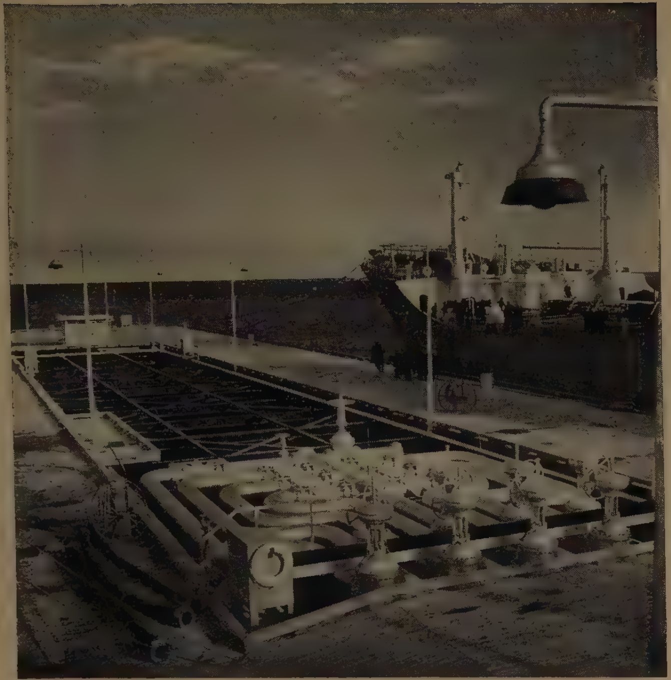

3"
1¼" dia.
Monel bearing pin
Cast-iron straight-face roller. Solid type, with fixed central hubs
Cap beam

ELEVATION AA

TYPICAL PIPE GUIDE

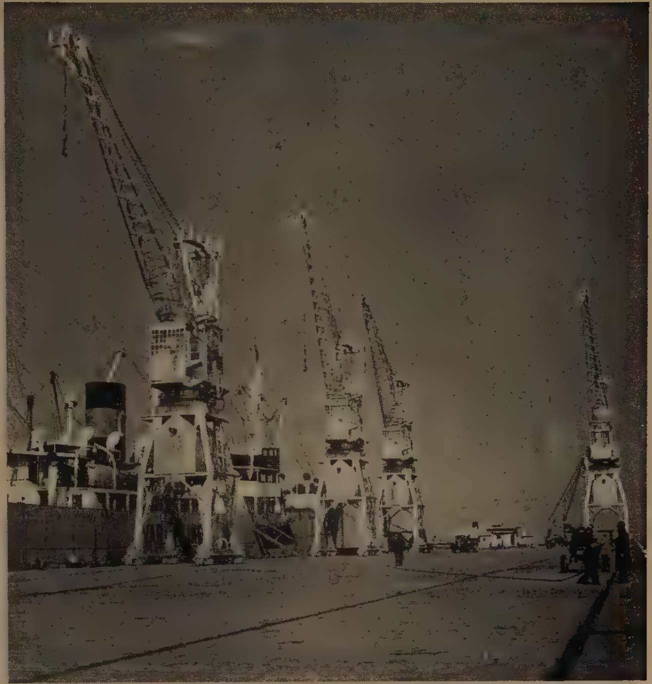
PIPE GUIDES

Fig. 28



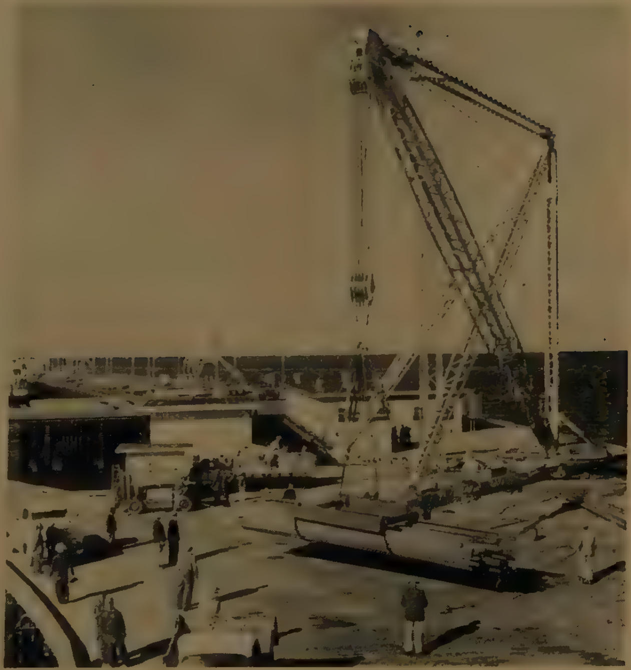
Oil Pier—Terminal Manifold

Fig. 30



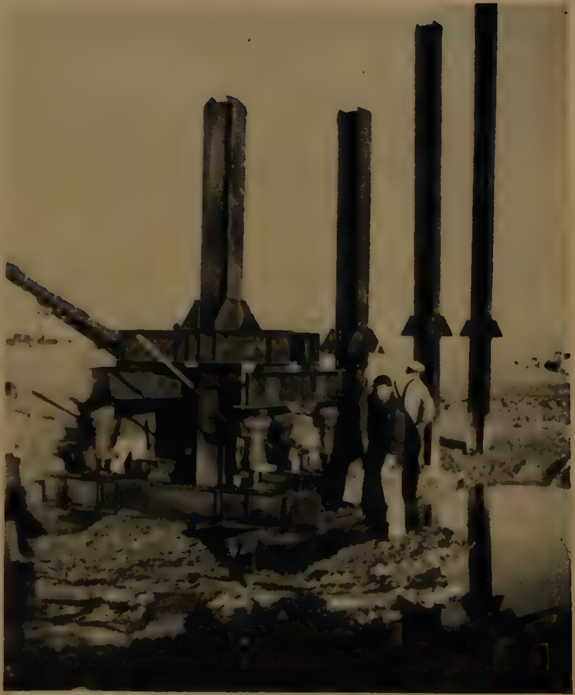
Cargo Pier—View of Completed Work

Fig. 31



Stiffleg Derrick carrying Test Load

Fig. 32



Test Piles Nos 16 to 19

Retaining straps installed on the approach pier constrain all lines from any tendency to leap out of the rollers, whilst on the oil pier the lines are supported direct on the cap beams. At all roller and cap-beam supports the larger pipes have rectangular $\frac{3}{8}$-inch-thick plates welded to their undersides, to take the wear resulting from movement. A single group of 10-foot-radius vertical expansion-bends has been introduced into each of the 24-inch lines between bents 2–80 and 2–83, each carried by a flexible spring support and fitted with an air-release valve at the crown. With a total movement of $5\frac{1}{2}$ inches the maximum stress developed in one of these bends is 11,250 lb. per square inch.

The design provides for movement of the terminal manifold caused by the differential thermal expansion of the pipes resulting from one or more being empty. To absorb this movement, steel double-bellows-type tied expansion joints are fitted, as shown in *Fig 28*, end movement being limited by the rods shown.

At each berth the oil-loading valves are located below deck-level but above a plated platform surmounted by a grating, the valve-operating wheels and pressure gauges being situated at deck-level. Figs 29, Plate 4, shows the arrangement. All valves are 20-inch-diameter double-disk parallel-seat cast-steel-gate-type and are rated at 300 lb. per square inch. By-passes fitted to each reduce the turning effort of opening by pressure equalization and valve stems are enclosed to exclude seawater and sand, position indicators being fitted above deck-level. It will be observed from Figs 29, Plate 4, that 20-inch-diameter branches are taken off the bottom of each 24-inch line and rest on rollers carried by hangers below deck-level.

Two branches supply a loading berth, each dividing into two 10-inch/12-inch-diameter branches at the platforms. Four 10-inch-diameter connexions thus serve the tankers at each loading point. Above deck-level the "crude" rising mains are fitted with 10-inch-diameter swing arms with 10/8 inch flanged reducers for use with 8-inch hoses, in 30-foot and 20-foot lengths; 6-inch-diameter hoses serve the bunker-fuel and oil connexions. At these points the loading pressure of the crude oil is about 150 lb. per square inch.

Provision is made for hose drainage, and for catching surplus oil, any spillage being returned to the shore by means of a compressed-air-driven salvage pump.

Hose Derricks

Figs 18 (facing p. 260) and *20* (facing p. 261) illustrate the twin air-operated derricks with 50-foot lattice masts for handling the oil-loading hoses located at each oil berth and mounted on a platform 30 feet by 15 feet and 17 feet above deck-level. The platforms extend over the roadways and are supported by portal frames welded to the pile caps.

Each derrick is capable of handling two 8-inch hoses of which there are four at each berth. These unusually long hoses (70 feet) are suspended during loading of a tanker from the derricks which also provide the adjustment necessary on account of the difference in level between the ship's deck and the pier. This difference sometimes amounts to as much as 25 feet.

CARGO HANDLING FACILITIES

Four 5-ton electric mobile portal cranes (see *Fig. 30*, facing p. 264) are available for use at either of the two berths. These operate on 15-foot-gauge track adjoining each face, the tracks being cross-connected by a shunting neck at the south end. A maximum wheel-load of 38,000 lb. is allowed for, the crane rails being set flush with the deck. This is arranged for by a two-beam bearer (see *Fig. 10*, facing p. 255) with the 6-inch flat-bottomed rail clipped to the connecting diaphragms.

A stiffleg derrick (*Fig. 31*) is provided for handling cargo that is too heavy for the portal cranes to lift. With a boom length of 100 feet and a capacity of 43 tons at 75 feet radius, the crane is able to pick up from the hold of all probable vessels.

Over an area of 90 feet by 20 feet in front of the derrick the deck structure has been strengthened to carry a uniform load of 1,200 lb. per square foot. To carry this additional load two intermediate rows of piles have been incorporated in this part of the pier.

EQUIPMENT

Lighting for the approach pier is provided by forty-eight standards, each supporting a 500-watt pendant street-type lighting fitting 20 feet above deck-level, and for the oil pier by sixty explosion-proof units with dome reflectors spaced at 90-foot centres, of similar power and fixing as for the approach-pier standards. The cargo pier is lighted by five 1,000 watt floodlights on each of two 75-foot towers placed at the north and south ends of the pier. Two additional 500-watt lights on the northern tower illuminate the deck in the neighbourhood of the stiffleg derrick, whilst three similar lights are carried by each portal crane.

Power is supplied from the Kuwait Oil Company's Mina al-Ahmadi Power Station at 3,300 volts, 50 cycles, 3-phase, and is stepped down to 440 volts for all motors other than those for the portal cranes, which operate at 220 volts, direct current.

The whole of the electrical equipment on the oil pier complies with the U.S. National Electric Code, Class I, Group D, for hazardous areas, whilst elsewhere this equipment is to weatherproof standards.

Fire protection is provided by an 8-inch-diameter ring main with sixty-nine hydrants spaced so that all areas can be reached with 100-foot hoses. Seawater is supplied by three deep-well pumps, delivering 1,000 gallons per minute at 300 feet head, driven by 100-horse-power vertical-shaft direct-connected motors, working off the 3,300-volt supply.

Pressure switches connected to the motor starters are arranged so that if a hydrant is opened the pressure drop causes the pumps to start.

An electrical alarm system gives audible warning in case of emergency. The system is controlled by push buttons at accessible spots on the oil and cargo piers which bring compressed-air-operated sirens into action.

Office and Storage Accommodation

A 32-foot-by-30-foot two-storey air-conditioned office building is situated midway along the oil pier. This includes offices for Pier Superintendent and Customs, telephone, storerooms, signalling siren, and searchlight for communicating with ships. A compressor house is also located adjacent to the vertical expansion-loops.

On the cargo pier, an air conditioned building, 107 feet by 38 feet, houses the Cargo Superintendent and Staff, also seamen's canteen, conveniences, and stores for pier operation and maintenance. Other buildings include a small transport office, a rectifier building for serving the portal cranes, and stores.

Services

Compressed air is required for operating the thirty-six motors for the oil-hose derricks, also fourteen motors for the operation of the return salvage pumps, and for the two ship's service points at each oil and cargo berth. To meet this demand, two vertical two-stage water-cooled compressors, rated at 300 cubic feet per minute, have been installed; they are driven by 75-horse-power squirrel-cage motors, which are automatically controlled by the air pressure in the receiver.

Fresh water still has to be imported occasionally. It arrives by small tanker and is discharged at Berths VII or VIII whence it is pumped ashore through a 6-inch-diameter steel pipe. Filtered and chlorinated water is supplied to the piers by a second 6-inch pipe for use thereon and for emergency supply through 2-inch metered connexions to ships.

Diesel and bunker-fuel mains are 12-inch- and 16-inch-diameter pipes respectively on the approach pier and oil pier, with 6-inch branch pipes to the cargo pier. Valves on the 6-inch pipe at the connexion points allow for maintenance shut-downs on the cargo pier while maintaining service to the oil berths.

The return salvage pipe is a 6-inch-diameter pipe, the lay-out being similar to that of the diesel and bunker fuel pipes. Salvaged oil is returned to a separator on shore.

Buoy-Type Oil-Loading Berths

These berths are equipped with an 8-inch flexible hose connected to ach of the pair of 12-inch steel pipe-lines which serve them. Each hose, which lies on the sea-bed when not in use, is attached to a buoy which, after the vessel is moored, is picked up by a line from the ship's boom and lifted on to the deck. Further bights on the line enable the latter to be hauled up sufficiently for coupling the hose to the deck oil-connexion.

BOAT HARBOUR

For the working of the port of Mina al-Ahmadi a boat harbour with a dredged water area of 20 acres, located at the south side of the shore end of the approach pier (see Fig. 3, Plate 1) is under construction. This harbour will serve the tugs, barges, launches, and ancillary craft in use and will also enable imported local materials to be discharged directly from lighters. It is provided with a lighter quay, 660 feet long, complete with a stiffleg derrick similar to that on the cargo pier. When completed there will be a tug berth with a depth of water alongside of 18 feet below L.W.O.S.T., whilst the remainder of the harbour will be dredged to a depth of 8 feet below L.W.O.S.T.

A marine engineering workshop with launch and barge slipways is also under construction adjacent to the harbour.

Part II.—Construction

by

R. C. Harvey, B.Sc.(Eng.), and J. W. Lowdon, B.Sc.(Eng.), MM.I.C.E.

INTRODUCTION

The decision to locate the new port at Mina al-Ahmadi was a courageous one. Speed of execution of the project was important. Inclement weather, with high temperatures and dust storms in summer, shortage of local engineering resources and skilled labour, and the fact that even drinking water had to be imported, added to the difficulties besetting the problem, but the employment of a group of American Contractors with extensive resources in the U.S.A., who were already operating in the Middle East and thus had experience of conditions similar to those prevailing at the site proved to be a wise choice.

The contractor, who was carrying out a number of other works, including the Boat Harbour adjacent to the approach pier, operated from a yard with workshops located ¼ mile from the beach and 1½ mile from the head of the approach. For taking materials to the work a small temporary loading wharf with a 15-ton derrick crane was constructed on the sheltered side of the south breakwater of the boat harbour and all the construction materials up to and including the deck timber were loaded on to barges here.

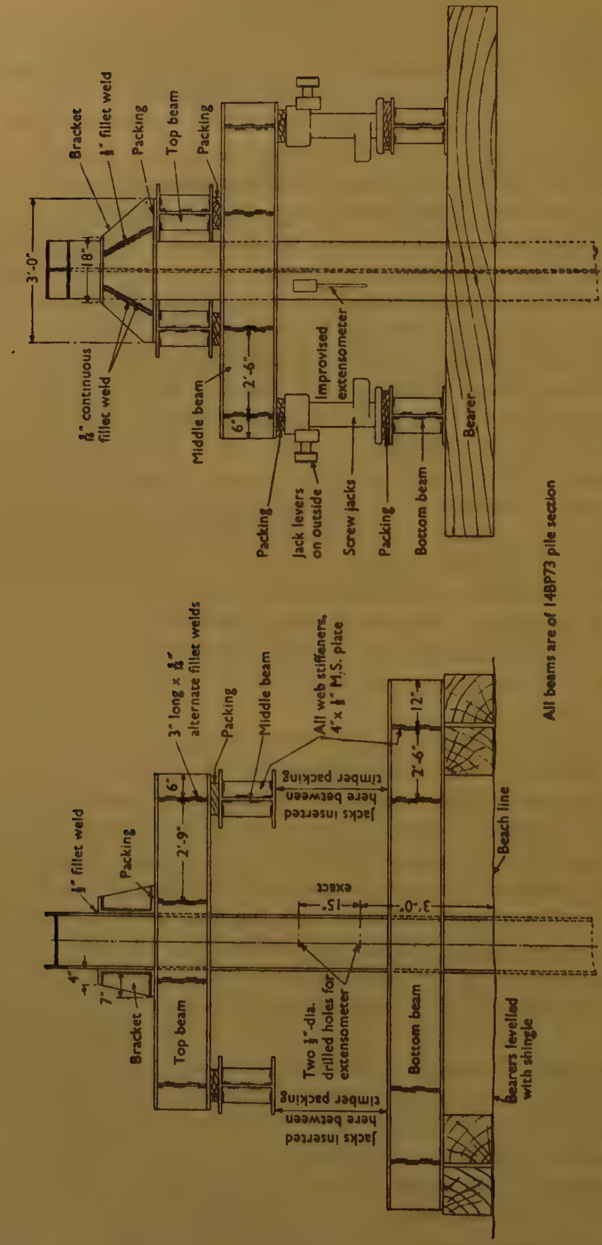
TEST PILES

The selected H-section pile (14BP73), referred to in Part I, is commonly used in America for bearing piles. It is rolled by the Grey process and this size is not yet produced outside the United States.

In order to ascertain the driving conditions, fifteen test piles, ranging from 45 to 80 feet long, were driven along the alignment of the permanent piling. Seven of them were located on the approach pier site, two on the cargo pier site, and six on the oil pier site, at approximate intervals of 500 feet. Driving was carried out by means of the floating pile frame which subsequently drove the permanent piles. This rig was equipped with a McKiernan-Terry 5S single-acting hammer with a striking member weighing 5,000 lb., falling 3·25 feet and delivering sixty blows per minute when worked by compressed air at 100 lb. per square inch. Driving the test piles took from the 23rd April until the 5th May, 1948, and penetrations ranging from 18 feet to 39 feet were obtained with calculated safe bearing values of not less than 74 tons. Five of these piles were subsequently subjected to a pulling test, the pull being exerted by a multi-purchase tackle attached to the pile and to the driving rig, the lift exerted being measured by means of an oil-drilling dynamometer reading up to 100,000 lb. It was only possible to exert the pull for short periods and attachment difficulties prevented a tension exceeding 35 tons being exerted on two of the piles. The remainder, however, resisted a pull which exceeded the maximum reading of the dynamometer. These piles were not coated with any protective material because the latter had not yet reached the site.

These tests emphasized the variations in the driving conditions of the sea bed, especially the high resistance of the compact calcareous sand and the toughness of the marine conglomerate. Subsequently during the course of the permanent-pile driving two of these piles were each re-driven by 300 blows of the hammer without moving them again. As permanent-pile driving proceeded the test piles were cut off at sea-bed level by blasting. From these piles it was concluded that the load-bearing value of the permanent piles would be completely satisfactory, but that, owing to the limited penetration at certain locations, further evidence was desirable regarding the grip which the sea bed would exert on the flexible structure. To obtain this, four uncoated piles, located at low-water line on the beach within the boat harbour, were driven to different penetrations at high tide by the floating rig. *Fig. 32* (facing p. 265) is a photograph of this group (numbered Test Piles 16 to 19) and *Figs 33* show, in detail, the arrangement for pulling them. Since no hydraulic jacks with reliable pressure gauges were available, 50-ton screw jacks with ratchet operation were employed and, for measuring the pull in the pile, an improvised extensometer was devised and made at site. This extensometer, which is illustrated in *Figs 34*, utilizes a dial gauge reading to one ten-thousandth of an inch for

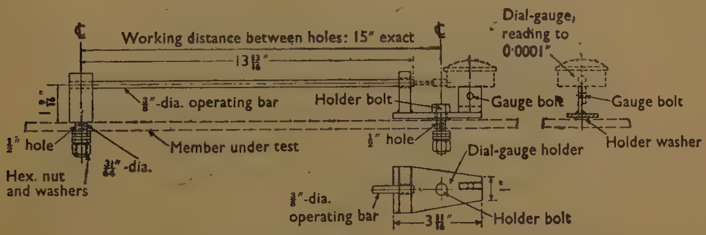
Figs 33



All beams are of 14BP73 pile section

Arrangement for Pulling Test—Piles Nos 16 to 19 measuring the extension on a length of 15 inches. It was bolted to the web of the pile under test midway between the ground level and the point of application of the load. The piles were driven on the 13th September, 1948, and pulled at low tides between the 21st and 31st December, that is, after a period too short for the final "seize up" of the piles to take place. The results of the test pull made on each of the four piles are shown graphically in *Figs 35* (*a*) to (*d*), which indicate the considerable increase in grip exerted by the compact calcareous sand as the penetration of the piles increased. *Fig. 35* (*e*) is a graphical driving record of test pile No. 19, showing the calculated driving resistance. This record is typical of the driving conditions in the material mentioned and is representative of an appreciable percentage of the permanent piles subsequently driven.

Figs 34



Scale: 1½ inch = 1 foot

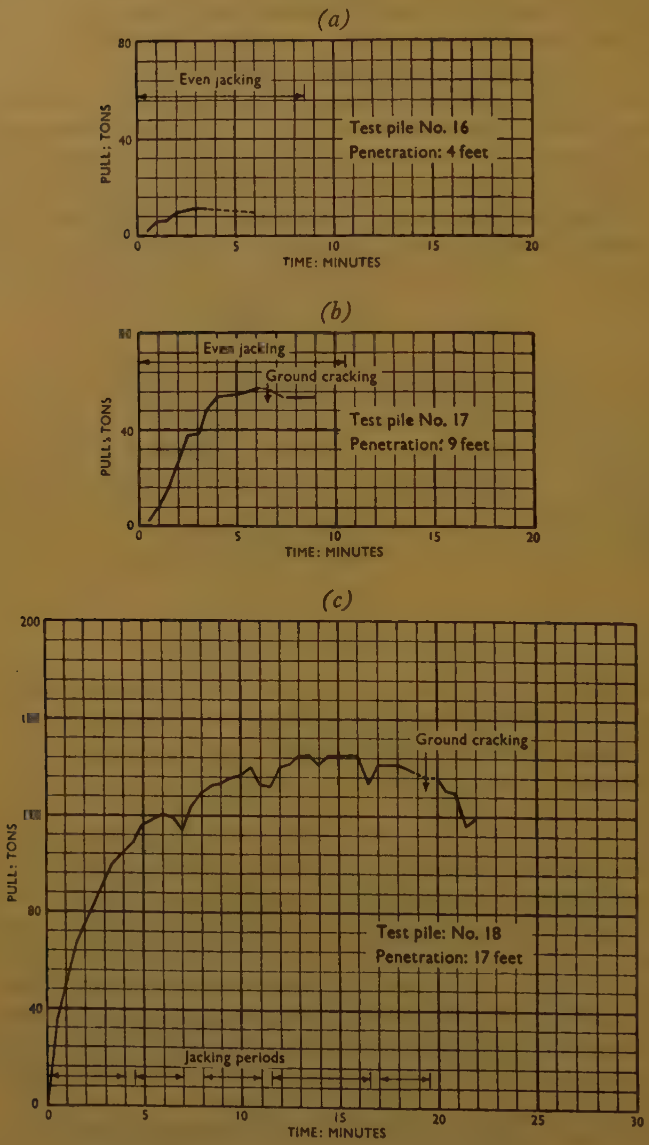
IMPROVISED EXTENSOMETER

The maximum resistance to uplift per foot of pile for the four depths of the piles in this group has been shown in the block diagram in *Fig. 35* (*f*). This illustrates the rapid increase in grip with depths exceeding 15 feet, for which the comparable increase in bearing resistance may be seen in *Fig. 35* (*e*).

Many hundreds of piles were to be driven for the permanent structure and it was, therefore, decided to conduct a further pulling test on a pile driven under easier conditions to an increased penetration. Permanent pile No. I-1-G, driven to a penetration of 36 feet at the shore end of the approach, well back on the beach in ground only partly wet was selected. It was coated with coal-tar enamel, except for the bottom 10 feet. The driving record as shown at the top of *Figs 36* is noticeably different from that of test pile No. 19, but the pulling record as shown in the lower diagram is somewhat similar. After the pulling test, this pile was re-driven by 200 blows and incorporated in the permanent structure.

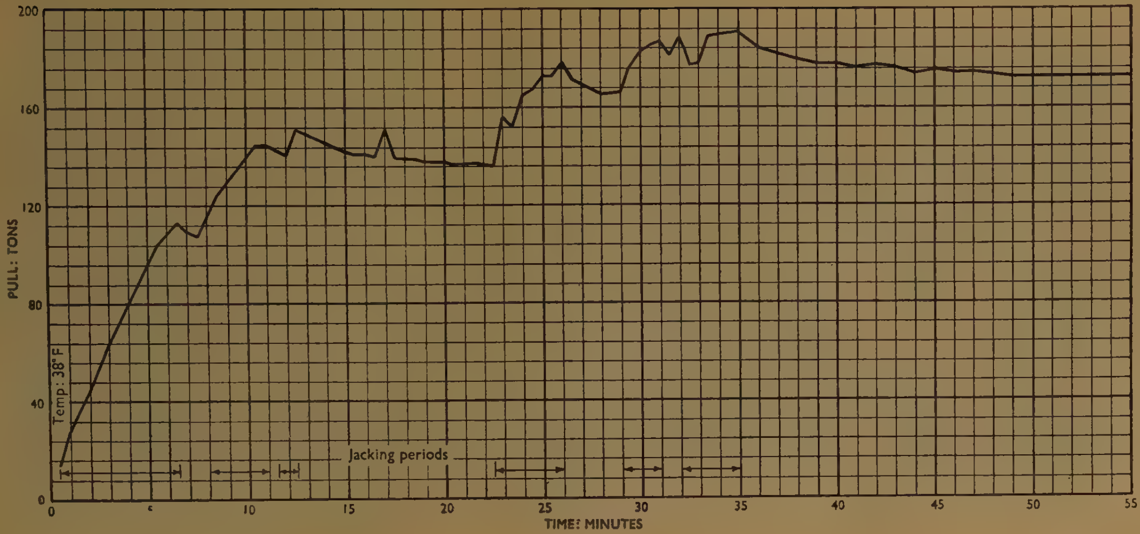
As mentioned in Part I, the design of the piers was based on the piles being fixed at a depth of 5 feet below the bed of the sea. In order to check this the opportunity was taken to make a deflexion test on a convenient permanent pile (No. I-10-F) driven for the portion of the pipeway adjacent to the north breakwater. *Fig. 37* (*a*) shows how this test was

Figs 35



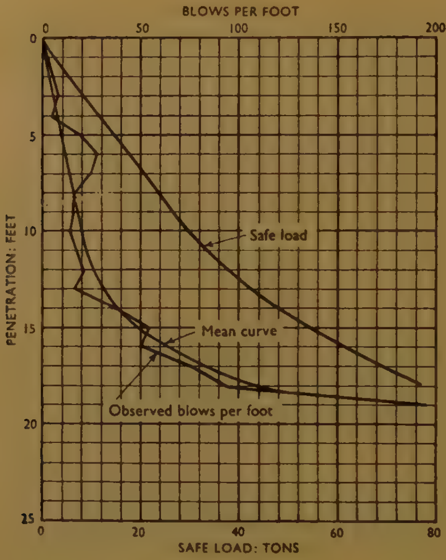
Pulling Tests on Test Piles Nos 16, 17, and 18

Fig. 35 (d)



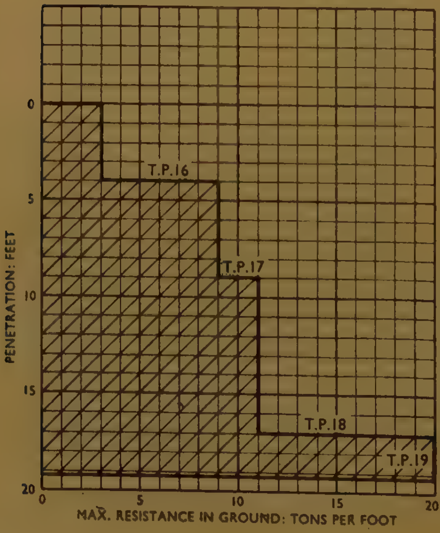
PULLING TEST ON TEST PILE No. 19 (PENETRATION 19 FT 3 INS)

Fig. 35 (e)



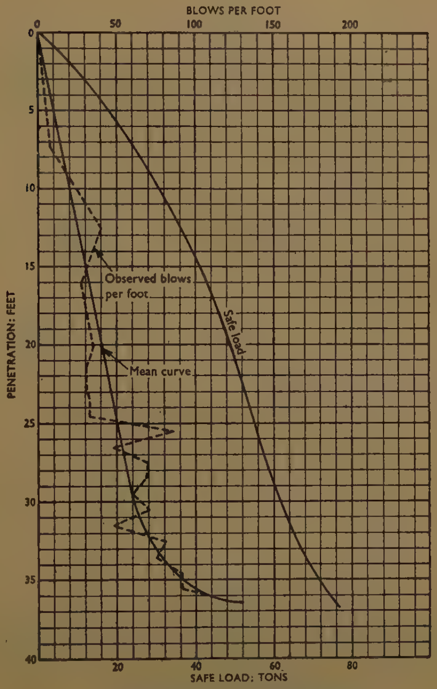
Driving Record for Test Pile No. 19

Fig. 35 (f)

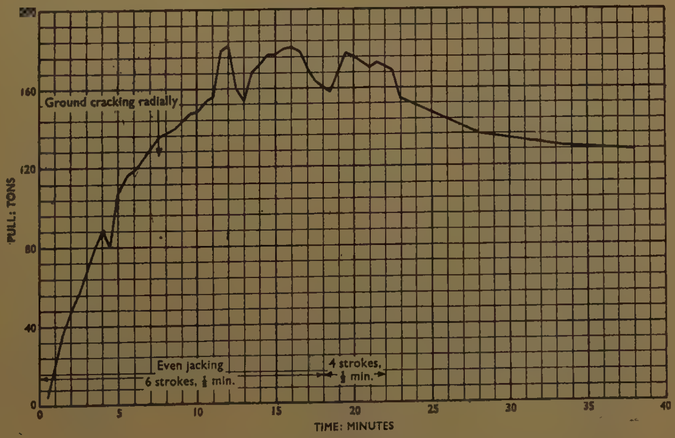


Relation between Pile Friction and Depth of Penetration for Test Pile No. 19

Figs 36



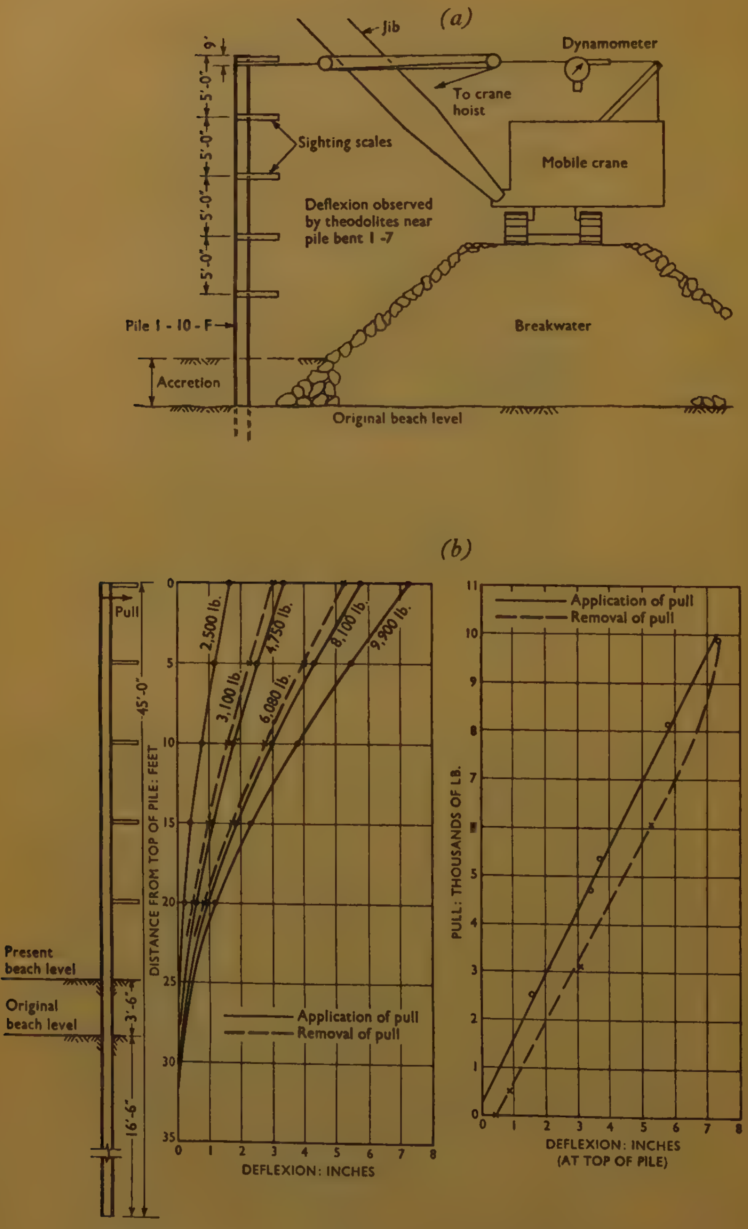
DRIVING RECORD AND SAFE LOAD CURVE



PULLING TEST

DRIVING AND PULLING RECORDS FOR PILE I-1-G

Figs 37



Deflexion Test on 14" × 14½" × 73-lb. H-Pile (No. I-10-F) carried out. Owing to the construction of the breakwater, the beach at the base of the pile had been built up by the accretion of sand to a height of 3 feet 6 inches, as shown in *Figs 37* (*b*). The initial driving set of the pile was 0·25 inch per blow and the final set, after 730 blows, was 0·05 inch. *Figs 37* (*b*) also show the recorded deflexion readings plotted to an exaggerated scale, indicating graphically that the point of fixity at all deflexions is not lower than 7 feet below the built-up sea-bed level. Calculation confirmed this. It was concluded that with a normal sea bed of firm strata the point of encastration would have been about 5 feet below.

Subsequently a group of test piles, Nos T.P.20 to 28 inclusive, each 90 feet long and covered with various protective coatings, was driven in deep water, adjacent to the site of the cargo pier. The object of this driving was to ascertain if various protective coatings, such as gunite, would stand up to severe driving conditions. This proved not to be the case and no positive information of any value was obtained from these test piles.

As construction proceeded, two extra H-section piles were driven between bents on the oil pier as observation piles. The intention is to extract one of these piles after a period of 10 years and the other after a period of 20 years in order to examine the underwater condition of the structure.

For more frequent examination of the underwater conditions two vertical 6-inch-by-8-inch broad-flange beam sections resting on the sea bed have been welded lightly to the structure at fender groups at the southern end of the cargo pier.

ANTI-CORROSIVE MEASURES

The measures to be taken to prevent corrosion of the structure presented a problem of some difficulty. Two somewhat similar structures constructed elsewhere in the Persian Gulf had been given no protection of any kind and had suffered severe corrosion. Cathodic protection was now being installed on these jetties, but its effectiveness was as yet unknown, nor was any experience available of the behaviour of conventional coatings in Gulf waters.

After extended enquiry, it was decided to coat all piling to within 10 feet of the toe and all structural members below the pile caps with one coat of coal-tar enamel, applied hot before driving or erection. The members above the pile caps, and the areas adjoining welded connexions were to be given two coats of coal-tar-base paint after erection. Steelwork above deck-level was to receive one coat of red lead and two coats of aluminium paint. On completion of the structure, cathodic protection was to be applied, initially by the use of magnesium anodes and at a later date by an external source of current.

The decision to omit coating from the lower end of the piling was based on experiments carried out with driven piles, which indicated that it was probable that the cost of the additional current required for the cathodic protection of this portion of the pile would be less than that of the coating. It was also thought that this length would suffer little corrosion. Coal-tar enamel was used on the piling and lower members and not on the remainder because the view was held that it was impracticable to apply the heated enamel satisfactorily in situ. The provision of conventional protection as well as cathodic was justified on the grounds of the heavier initial cost of the latter were no coating applied and some doubts of its complete effectiveness on so extensive a structure.

MARINE LIFE

An appreciation of the protective measures necessary for a structure of this sort is not possible without some reference to the marine life found in the Persian Gulf. During September 1948, the piles already driven at Mina al-Ahmadi were subjected to a very active attack by acorn barnacles. At this time of the year the seawater temperature is at its maximum and this low form of life is more prolific than at other seasons. It was found that small barnacles with a basal diameter usually not exceeding 12 millimetres settled on the piles between low-water and half-tide lines. The density of settlement was so great and the barnacles so active that patches of the enamel coating were lifted from the pile's surface exposing the steel beneath. This species of barnacle, which is pinkish white in colour and which develops second and third colonies within a matter of weeks was identified as *Balanus amphitrite*—variety *denticulata*, first described by Brock in 1927 in the "Results of a Cambridge Expedition in the Suez Canal in 1924." This concentrated barnacle attack caused some alarm and the driving of permanent piles was stopped for 7 weeks during which time the situation was studied. It was finally decided that with more experience and skilful application of the coal tar enamel and the application of cathodic protection the corrosion of the piling both under-water and in the lower half of the tidal range would be prevented. Subsequent examination of the structure has confirmed this. Another common tropical barnacle, *Balanus tintinnabulum* settles on the piles below low water, but with less density. In addition, oysters and other bivalve molluscs, calcareous tube worms, and polyzoa are prolific. The marine conglomerate which is so common on or immediately beneath the sea bed at many places in the Gulf consists largely of the shells of these forms of life, sometimes cemented together when in an active state. Marine botanical growth is also prolific in these waters and after a few months' immersion any pile, whether wood, steel, or concrete, resembles the wall in an aquarium. The presence of *Teredo navalis* was verified. Biological investigation in connexion with marine growth as it affects the maintenance of the structure is being continued.

PILE COATING

Before hot coal-tar enamel can be applied to steelwork, very thorough cleaning and the application of primer is necessary. For cleaning, the contractor installed steel-shot blasting equipment in his yard. This comprised three machines feeding hand-operated nozzles for cleaning the different faces of a steel H-pile as it was slowly pulled through two lighted steel chambers, through the windows of which the operators viewed their work. The steel-shot blasting cleaned the steel to a silver-grey finish but the apparatus was not proof against the rigours of the climate and it was found that extensive maintenance was necessary; furthermore, the shot consumption, in spite of the recovery, was very heavy. High humidity in winter and the trace of lubricating oil which at times was carried over by the compressed air both interfered with the feeding of the shot and left a fine film of oil, dust, and condensate on the piles, which had to be cleaned off with petrol before the coal-tar primer could be applied. The average rate of feeding through the machine was 9 inches per minute giving a maximum output of eight 90-foot-long piles per day. In September 1948, the use of shot-blasting equipment was stopped and eight sand-blasting units were installed instead. Provided that it was properly screened and dried the quarry sand used gave a thoroughly clean surface, even in the humid winter months, and the maintenance of the apparatus was much less troublesome. For drying the sand the device shown in *Fig. 38* was made locally.

At the outset after priming, hot coal-tar enamel was applied to the piles and steelwork by means of the special brushes which are used for the application of such compounds. However, because of the limited temperature range (450° to 475° F.) within which it could be brushed, great difficulty was experienced in getting a continuous coating with a thickness between $\frac{3}{32}$ inch and $\frac{1}{16}$ inch, as had been specified. After a great deal of experimenting it was found that the most effective way to apply the enamel, under the conditions prevailing and with the labour available, was to pour it. It was found that the desired thickness could be obtained when the enamel was poured on to a surface with a slope of approximately 1 in 4, the surplus running off. The waste, which amounted to about 30 per cent of the total enamel used, was scraped off the ground and collected in pits for subsequent use on less important structures.

After cleaning, the H-section piles were placed on bearers for welding and were spliced to the desired length, pointed, and provided with reinforced heads as necessary, and then transferred to the coating yard. This yard which was laid out on a level piece of ground had flat-footed rails fixed on bearers at 20-foot intervals. Each pile was held in three rollers (see *Fig. 39*) which enabled it to be rolled along as work proceeded. The final procedure for the coating was that the night shift cut up the required number of drums of enamel by means of a power winch and wire rope, filled up and lit the heating boilers and brought the enamel to a temperature of 450° F. ready for use by the day shift. Dial thermometers were used to control the temperature. Twenty 90- or 100-foot-long piles were coated in two hours, after which the buckets and boilers were burned out and the priming of the piles for the next day's work undertaken. *Fig. 40* is a photograph of the pouring of the enamel on to the piles. A baffle device was used to prevent drip from the upper edge of the sloping flange on to the web. The coating of the web and the inner sides of the flanges was effected by a fixed number of men stationed at equal distances alongside the pile with the required amount of enamel in pouring buckets. At a given signal from the foreman each man emptied his bucket of hot enamel on to the bottom flange. The pile was then rolled first in one direction to cover the edge of the bottom flange and then in the other direction to cover the side of the web and the other flange and onwards to decant the surplus enamel on to the ground. The Arab workmen employed on this work were able to produce uniformly coated piles with a measured thickness between the specified limits. The adhesion of the enamel poured in this way at the correct temperature on a properly applied primer was good. The four main factors controlling the satisfactory application of this difficult protective material are, in the Authors' opinion :—

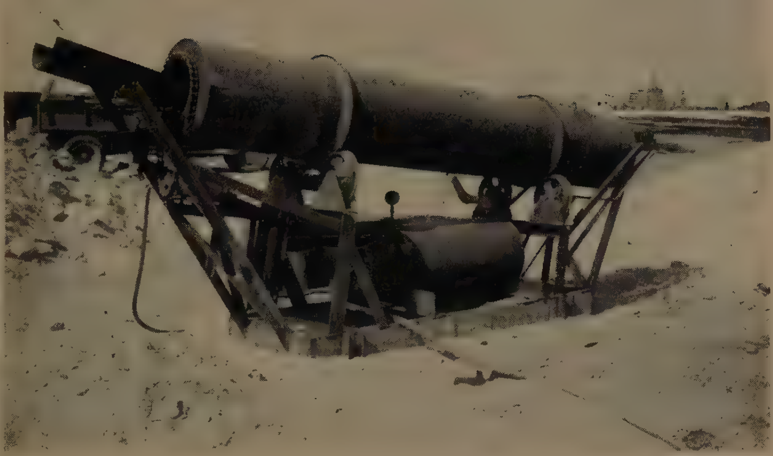
(1) Sand blasting the steel until it is a uniform grey colour.
(2) Careful application of the primer.
(3) Maintenance of the correct temperature of the enamel at the time of pouring.
(4) Adjustment or manipulation of the member being coated to the correct slope for the desired thickness.

Since all piles had to be cut off for capping after driving, the top 4 feet of each was not coated.

Cathodic Protection

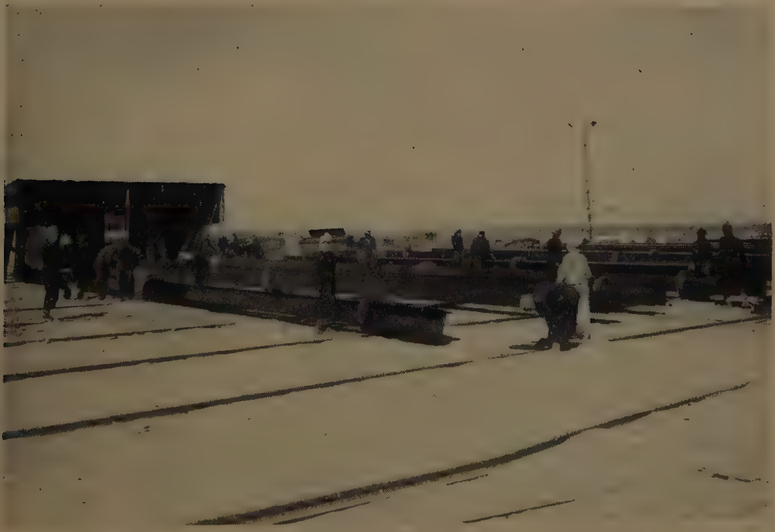
The object of cathodic protection is to prevent the electro-chemical corrosion of a metallic structure located in the earth or in seawater by means of superimposed electrolytic action. This form of protection is common practice in America but as yet is not used extensively by British engineers. In application, the arrangement may be that of the primary or electrolytic cell with the steel structure forming the cathode (or negative pole), the surrounding seawater or moist earth being the electrolyte, whilst the anode (or positive pole) is formed by a mass or masses of another metal (say zinc or magnesium), higher in the electro-chemical scale, and connected to the structure by a suitably insulated cable.

Fig. 38



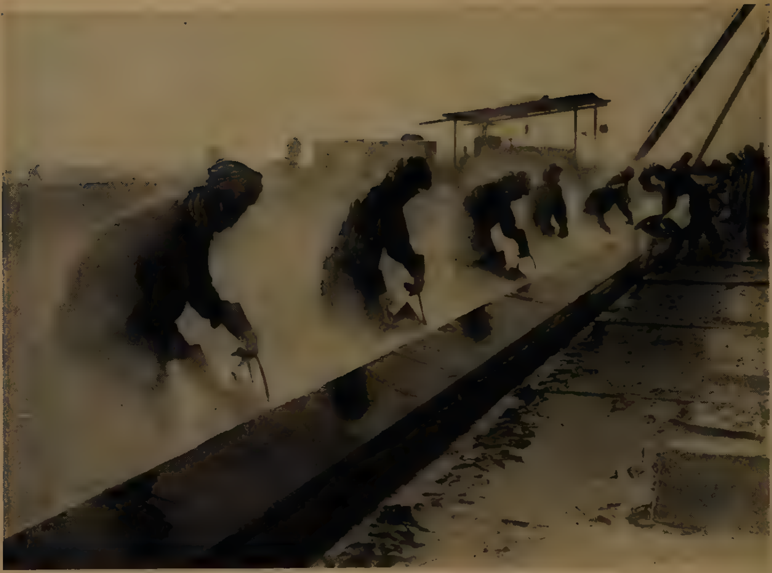
Improvised Sand Drier

Fig. 39



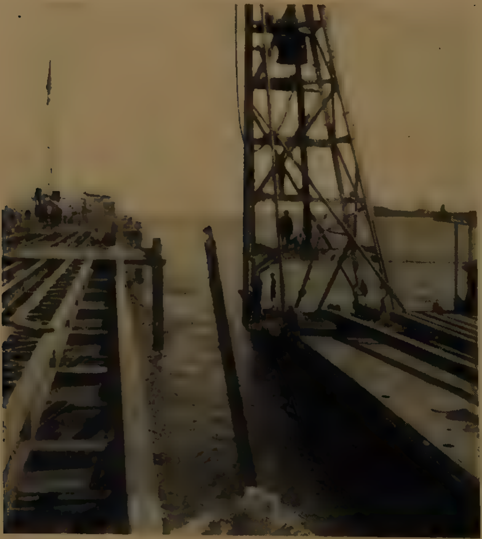
Pile Coating Yard

Fig. 40



Enamel Pouring

Fig. 42



Pitching a Fender Pile

The protective current slowly reduces the anode and a protective deposit is built up on the cathode or structure, depending on the composition of the anode and the surrounding electrolyte.

Alternatively a mass of scrap metal or carbon is used to form the anode in which case the flow of hydrogen ions to the cathode is induced by a suitable low-voltage direct-current supply.

On the piers under consideration, the installation of the first stage of cathodic protection using magnesium anodes was begun in September 1949, and completed in January 1950; that is to say, it followed within a few weeks of construction. Initially it consisted of fourteen 50 lb. and two hundred and seventy 200 lb. magnesium anodes, the composition specified being as follows:—

Aluminium:	5·3–6·7 per cent
Manganese:	0·15 per cent, minimum
Zinc:	2·5–3·5 per cent
Silicon (max.):	0·3 per cent
Copper (max.):	0·05 ,, ,,
Nickel (max.):	0·002 per cent
Total other impurities:	0·3 ,, ,,
Magnesium:	remainder

All anodes were positioned on the sea bed, a connecting wire being suspended from a spool insulator immediately under the decking, then running to a junction box for the insertion of a resistance to control the current flowing to designed limits, and finally earthed to the structure. The final distribution of anodes and method of suspension is illustrated in Figs 41, Plate 4.

Spacing was decided on after observing the behaviour of the first anodes placed and provided an initial current of approximately 0·96 milliampere per square foot of surface protected at 0·3 volt between anode and structure or 0·45 ampere per pile; the total current being about 1,800 amperes. The corresponding figures after polarization of the steel were 0·40 milliampere, 0·40 volt, and 670 amperes. One lb. of magnesium of the composition described has been found to give approximately 500 ampere-hours and the effective life of the initial protection is estimated to be about 2 years.

The protection afforded has in general exceeded expectation, in that protection has been given in the tidal range as high as +6·00 or +7·00. After one years' operation it was found that current density fell at high tide owing to the deterioration of the coating in the barnacle range and one hundred additional 200-lb. anodes were installed to provide the additional protection required.

Test pile TP41, sited at gravity fender No. 41 at the south end of the cargo pier was removed for examination in September 1950, and showed full corrosion-mitigation up to +6·00 with very slight corrosion above this level. At points on the pile where the coating had been damaged below this level during the installation, the steel was uncorroded and retained the original sand-blast pattern.

A "forced drainage" system of protection is now being installed to replace the present magnesium anodes. This will consist of sixteen metal-plate rectifiers, each giving an output of 150 amperes at 50 volts to sacrificial steel anodes. Two or three anodes of about 8 tons weight will be used for the approach pier; for the oil and cargo piers a larger number of anodes, each of about 1 ton, will be located among the piling so that damage to the anode cables by the anchors of ships alongside may be avoided. It is anticipated that the annual cost of protection by this system will be about one-quarter of that of the present magnesium anodes.

PILE DRIVING

With the exception of the piles for the first 500 feet of the pipeway at the shore end, all the piles on the approach pier, oil pier, and cargo pier were driven by the same floating pile-frame as was used for the test piles.

This very useful piece of equipment consisted of a 70-foot timber frame mounted at the end of a barge 100 feet long, 29 feet wide, and 8 feet deep. The end supporting the frame was vertical but the other was of the swim type. The pile frame (*Fig. 42*) was reinforced by pipe struts welded to the deck and, in accordance with American practice, inside leaders capable of taking the H-piles were used. The hammer, with the forged steel driving-head attached, was hung within these leaders. Two air compressors, each of 500 cubic feet capacity, were located below the deck and supplied the air required for driving the hammer, whilst the three triple-drum winches, one for the frame and two for the anchor, each had its own high-speed diesel engine. The barge was ballasted with sand, capped with concrete, and had a working draught of 4 feet 9 inches. It rode small swells and choppy seas quite well and it was necessary to bring it into the shelter of the harbour only at times of storms.

Fig. 43 illustrates the arrangement of this floating pile-rig when driving. Coated piles ready for driving were supplied by means of decked barges which were handled by sea-mule tugs. The method of pitching a pile, which was both rapid and impressive, was to move the rig with its attached barge back a distance of 20 feet from the pile just driven, pull the bottom end of the next pile on the barge to the end of the outrigger beam shown in the diagram and at the same time lift with the slinging rope fastened to the upper end of the pile. Skilful operation of these two pulls swung the pile both outwards and upwards and the lower end splashed into the sea and the pile swung into the leaders (see *Fig. 42*). The rig was then moved sideways and forward on to the approximate location for the pile which was lowered so that the point was 2 feet above the sea bed. By sighting with a theodolite placed on the framed portion of the jetty and measuring with a steel wire from the last row of braced piles, the new pile's position was

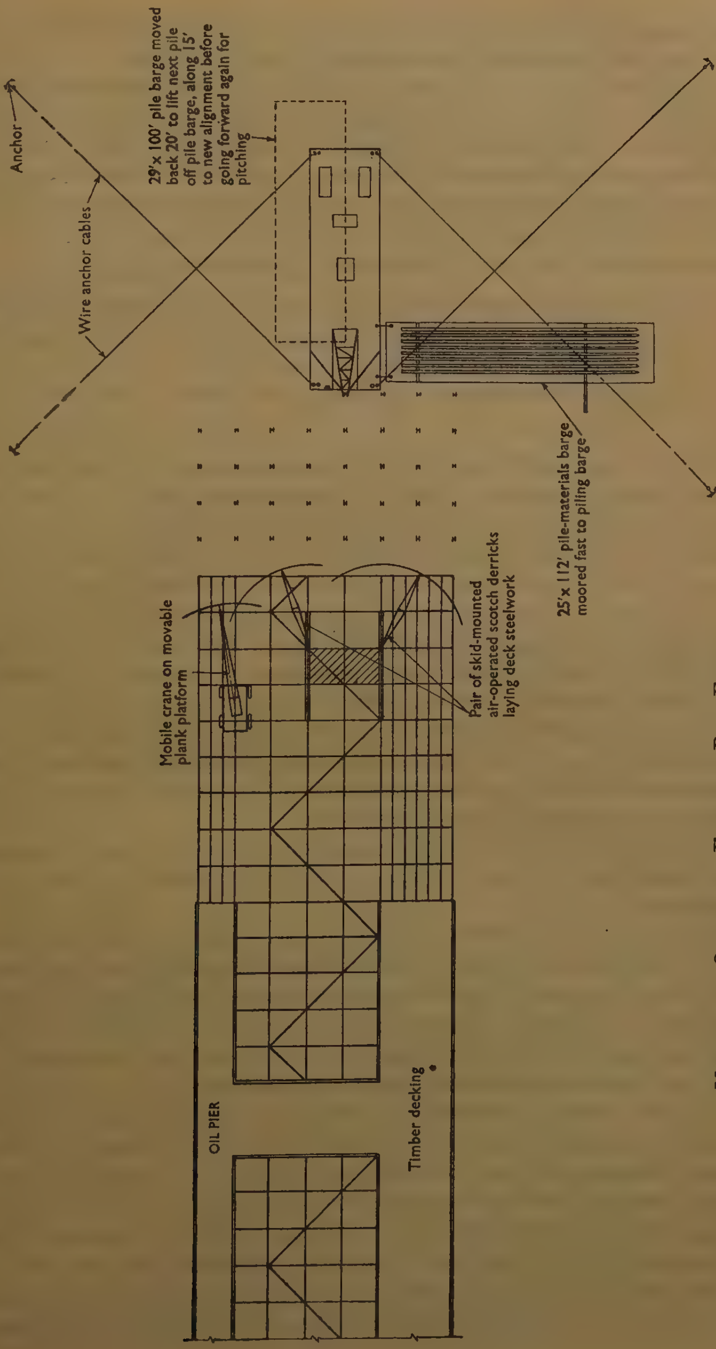

Anchor
Wire anchor cables
29'x 100' pile barge moved back 20' to lift next pile off pile barge, along 15' to new alignment before going forward again for pitching
25'x 112' pile-materials barge moored fast to piling barge
Mobile crane on movable plank platform
Pair of skid-mounted air-operated scotch derricks laying deck steelwork
OIL PIER
Timber decking

Fig. 43

METHOD OF OPERATING FLOATING PILE FRAME

located and the pitching completed by dropping it on to the sea bed. The hammer was immediately lowered and driving commenced.

At the shore end of the approach pier the specified penetration into the sea bed was 15 feet. This was increased to 20 feet at the sea end but on the oil and cargo piers a penetration of 25 feet was asked for. The tough strata under the sea bed, however, prevented this depth from being obtained in many zones and, as a result of the tests made on test piles Nos 16 to 19, a reduced penetration was accepted as satisfactory. Driving conditions were, in general, found to be of three types :—

First, steady resistance increasing with depth—similar to pile I-1-G.
Secondly, hard conglomerate sea bed followed by soft driving before increased resistance was obtained.
Thirdly, soft driving into the sand of the sea bed, with hard driving after about 15 feet.

For the first condition, piles with square toes and driving heads were driven without difficulty. When the conglomerate on the sea bed had to be penetrated, the piles were pointed, as shown in *Figs 44.* It was found, however, that a thickness of more than 18 inches caused a number of piles to split at the toe where the flange joined the web. *Fig. 45* (facing p. 286) shows the damaged toe of such a pile which had been lifted for burning-off the damaged toe and re-driving. To overcome this difficulty the welded 6-inch-by-4-inch-angle reinforcement shown in *Figs 44* was introduced and found to be very effective.

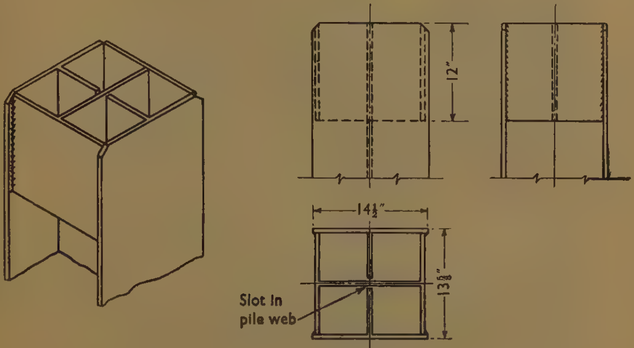
For the hard-driving conditions, the head of the pile also needed strengthening and this was economically effected by welding 12-inch pieces of cut-off H-pile material as shown at the top of *Figs 44* to make what was known at site as a " waffle head."

Piles at the shore end of the approach pier were driven by means of leaders hanging from the jib of a mobile crane working on the adjacent breakwater.

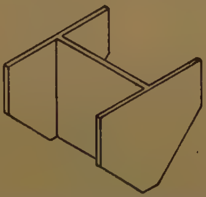
Driving of the approach-pier piles was completed on the 9th January, 1949, after which the piles for the first three bents of the oil pier were set out, using a temporary theodolite station located on the group of test piles referred to previously. These three bents were then decked and framed and the braced structure was used for setting out the remainder of the piles for the oil pier and subsequently for the cargo pier.

In good weather the floating rig was able to drive ten piles a day without difficulty but there were many delays owing to bad weather, the worst storm occurring on the 16th, 17th, and 18th of January, 1949, when a crane barge used for the erection of deck steelwork foundered (without loss of life) after it had been taken to sea by a tug. At the northern end of the oil pier, where driving conditions were easier, the maximum number of piles driven per day of 12 hours was forty. This is an outstanding performance even in ground of this nature and, in the Authors' opinion, could only be obtained with steel piles of this type. A spell of fine weather at the end of 1949 enabled the 1,211 piles of the cargo pier to be driven by one shift at an average of ten piles a day, including all stops.

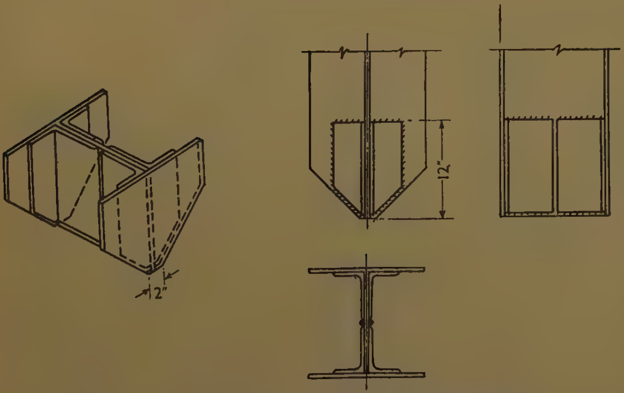
Figs 44



REINFORCED PILE HEAD



TOE OF PILE FOR MEDIUM DRIVING



TOE OF PILE REINFORCED FOR HARD DRIVING

Pile Head and Toe Reinforcement

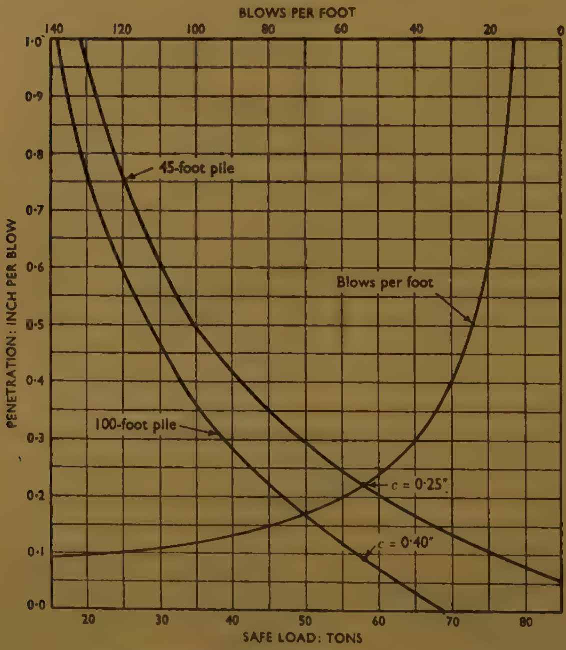
In order to ascertain if any piles were damaged by the hard driving and if the coating was in good condition, approximately 60 per cent of all the piles were inspected underwater. In no case was the enamel found to be damaged at sea-bed level.

The following working loads were specified for the various piles :—

Approach pier . . .	58 tons
Oil pier	36 ,,
Cargo pier	45 ,,
Gravity fenders . . .	58 ,,

Fig. 46 shows safe working loads for piles 45 feet and 100 feet long.

Fig. 46



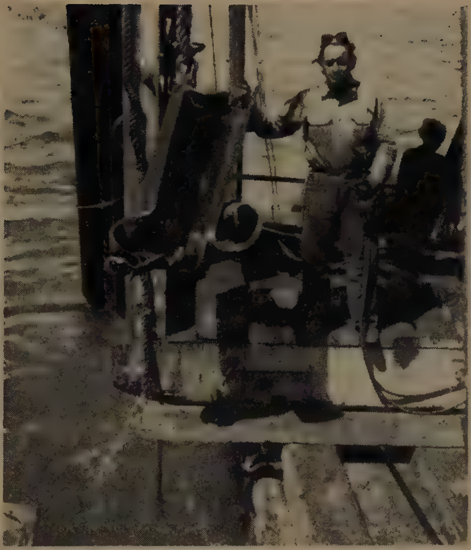
Safe Working Loads for 14 BP73 Piles

McKiernan-Terry single-acting hammer S-5
Weight of striking part : 5,000 lb.
Length of stroke : 3·25 feet
Factor of safety : 2·5 (Hiley formula)

These were calculated by the Hiley formula, using a factor of safety of 2·5 on the ultimate resistance to penetration. The temporary quake " *c* " of the pile and ground employed in these curves is 0·25 inch and 0·40 inch respectively for driving conditions corresponding to the specified load of 58 tons.

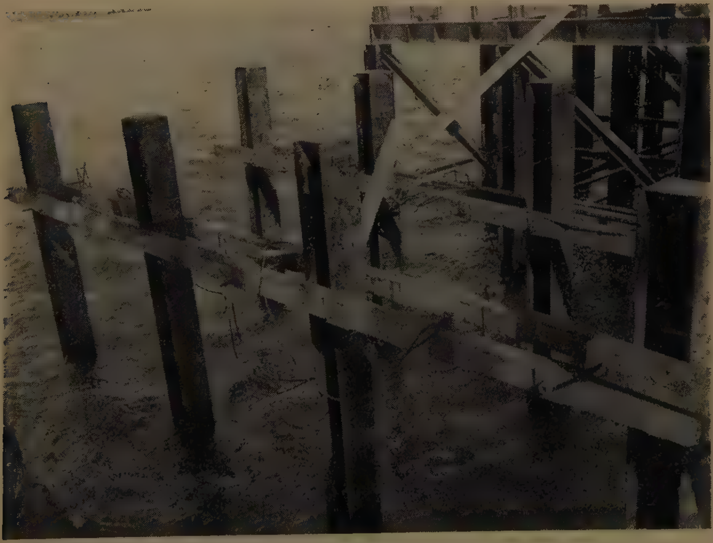
The curve on the right of the figure gives the number of blows per foot for

Fig. 45



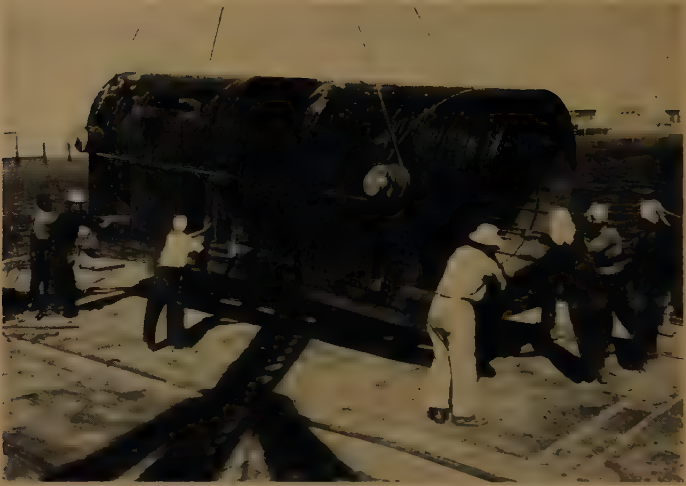
DAMAGED PILE TOE

Fig. 48



PILE CAPPING CLAMP

Fig. 50



Fender Cylinder

Fig. 51

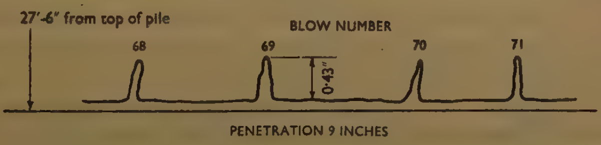
Erection of Fender Cylinder

any desired set, because it was found convenient on the work to specify the number of blows required for the last foot of penetration and not the actual set in inches, thus obviating the difficulties arising from any surge of the rig. A driving record was taken for every pile, which was a very satisfactory arrangement because it enabled the supervising staff to appreciate what was happening with the minimum of difficulty and misunderstanding.

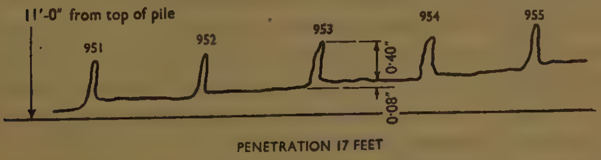
The value of "*c*" of 0·25 inch for 45-foot piles was based on quake records taken early on during the construction of the approach pier on a 45-foot-long pile of the pipeway. Since it was impossible to obtain quake records for 100-foot-long piles from the floating pile-rig a quake value of 0·40 inch was assumed for those piles. Towards the end of the work, when the 100-foot-long fender piles were driven, records were obtained at two of them; one of these records is given in *Figs. 47*. From this it will be seen that "*c*" is nearly equal to the assumed value of 0·40 inch when the set is 0·08 inch.

The 100-foot piles were very flexible and for hard driving conditions this length should not be exceeded but in-situ splicing adopted.

Figs 47



(Pile broke through 3 feet of conglomerate after 434 blows)



Pile Quake Record for East Fender Pile (100 feet long) at Bent 3–85

Erection of Steelwork

The flexibility desired in the structure permitted all bracing to be above L.W.O.S.T., the lowest in the vicinity of each group of fenders being at + 4·00. The next bracing was located at level +7·00, whilst the continuous longitudinal bracing was at +12·00. This arrangement also facilitated the complete welding of the structure which had to be carried out above water.

All steel members arrived at site fabricated. The bracing and stiffening members located below pile-cap level were cleaned, primed, and coated with enamel in the yard, leaving bare the ends and other places where welding had to be done. The method of capping the piles and erecting the deck steelwork was as follows.

Three or four adjacent piles of a bent were held together at the correct spacing of 15 feet by means of the double-beam clamps illustrated in *Fig. 48* (facing p. 286). The piles were then cut off by oxy-acetylene flame at the required level and the pile caps, to which the lugs had already been welded in the yard, were put into position and welded on. The pile-cap beams were then lifted on by a mobile or skid crane located on the adjacent framed structure and bolted; subsequently the stringers were placed in position and tack-welded. Along the middle of the oil pier and the cargo pier, the longitudinal and diagonal members of the horizontal-truss systems were erected before the superimposed stringers were placed. The longitudinal and diagonal bracing was then clamped in position and tack-welded, it being an easy matter to adjust the structure to the alignment required. All these members fitted remarkably well on account of the accuracy with which the piles had been pitched and their flexibility. The remainder of the welding of the cap beams, stiffening girder system, and stringers was then carried out, after which the welding of the bracing was finished.

All steelwork above pile-cap level was erected as received from the shops and was subsequently sand-blasted and given the two coats of coal-tar-base paint. Where the enamel on piles or other members had to be removed for welding, this was done by means of small pneumatic chipping chisels before cleaning with wire brushes. *Figs 10* and *17* (facing p. 255) are general views showing the erection of the steelwork.

The progress of steel erection followed the pile driving very closely, except at one part of the approach pier when the late arrival of a number of cap beams held up part of the work. Here it was found that the rough weather prevailing at the time caused the piles to sway too much, with the possibility of buckling at sea-bed level, and temporary caps were, therefore, provided by utilizing piling material.

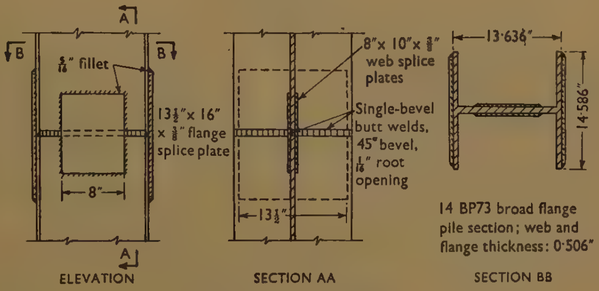
WELDING

The electric arc welding of the structure was carried out in accordance with American practice for structural steelwork, using coated electrodes of $\frac{3}{16}$ inch and $\frac{1}{4}$ inch diameter. The same type of electrodes was used for welding in all positions. Single-welder portable D.C. generators, driven by petrol or diesel engines, were used in the contractor's yard and on the work. In the yard, the welding was confined to splicing and reinforcing piles where necessary, the partial assembly of pile caps, the gravity fender hangers, pipe specials, and certain small assemblies.

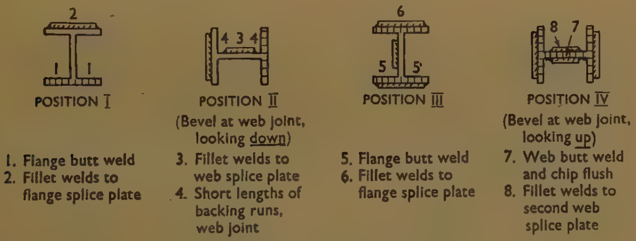
Of all these tasks the splicing of piles was the most important. *Figs 49* (*a*) give the details of a pile splice. At the beginning of the work, however, it was found that the butt welding of the webs was liable to crack but, by altering the procedure as shown in *Figs 49* (*a*) this cracking was completely cured. This work was carried out on bearers and the piles were turned so that all welding was done either in the down-hand or vertical position.

For the site connexions, the members were positioned by tack welding by one party of welders; other welders followed after an interval and

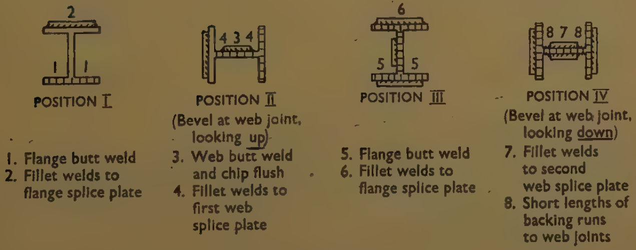
Figs 49 (*a*)


A
B
B
5/16" fillet
13½" x 16" x ⅝" flange splice plate
8"
A
ELEVATION
8" x 10" x ⅜" web splice plates
Single-bevel butt welds, 45° bevel, 1/16" root opening
13½"
SECTION AA
13·636"
14·586"
14 BP73 broad flange pile section; web and flange thickness: 0·506"
SECTION BB

DETAILS OF PILE SPLICE
Scale: ⅛ inch = 1 foot


2
1 1
POSITION I
1. Flange butt weld
2. Fillet welds to flange splice plate
4 3 4
POSITION II
(Bevel at web joint, looking down)
3. Fillet welds to web splice plate
4. Short lengths of backing runs, web joint
6
5 5
POSITION III
5. Flange butt weld
6. Fillet welds to flange splice plate
8 7
POSITION IV
(Bevel at web joint, looking up)
7. Web butt weld and chip flush
8. Fillet welds to second web splice plate

ORIGINAL WELDING PROCEDURE
(Cracking starts in welds No. 4)

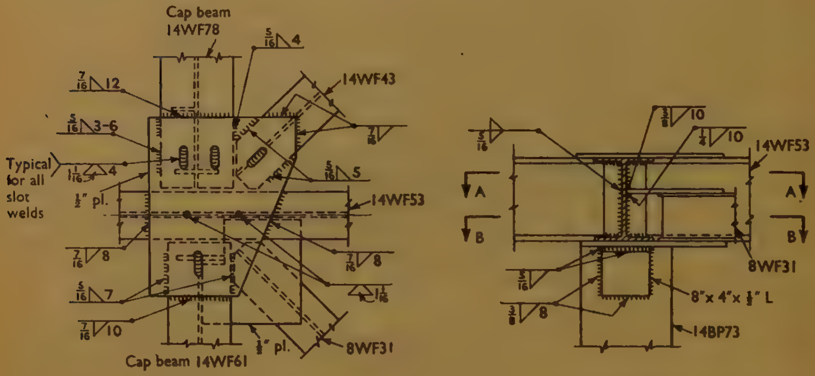

2
1 1
POSITION I
1. Flange butt weld
2. Fillet welds to flange splice plate
4 3 4
POSITION II
(Bevel at web joint, looking up)
3. Web butt weld and chip flush
4. Fillet welds to first web splice plate
6
5 5
POSITION III
5. Flange butt weld
6. Fillet welds to flange splice plate
8 7 8
POSITION IV
(Bevel at web joint, looking down)
7. Fillet welds to second web splice plate
8. Short lengths of backing runs to web joints

REVISED WELDING PROCEDURE
(No cracking)

H-PILE SPLICE

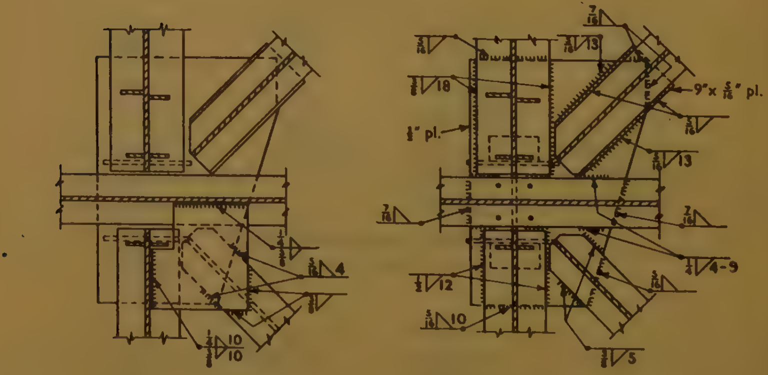
completed the welding in accordance with the drawings. Some of the latter work was done by night shifts. *Figs 49* (*b*) show details of one of the panel-point joints of the horizontal truss on the cargo pier.

Figs 49 (*b*)



PLAN

ELEVATION



SECTION AA

SECTION BB

Scale: ⅛" = 1 foot

Cargo Pier Horizontal Truss. Typical Welded Joint

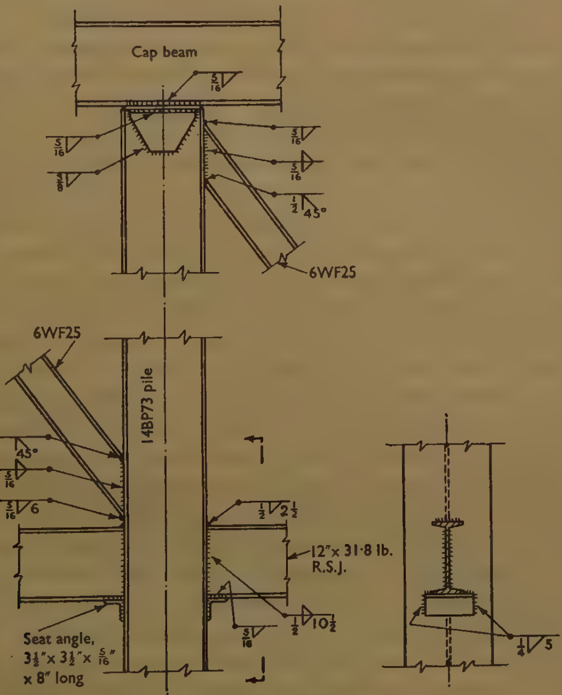
(The welding symbols used in these Figures are in accordance with American practice. A similar scheme of symbols is now incorporated in British Standard No. 499)

It was found that the carbon content of some of the steel was more than 0·25 per cent, which is rather high for welding, but the point was given particular attention by the inspecting and testing engineers at the producing end.

All welding was carefully inspected and very little of the work had to be cut out. Samples were periodically removed from joints and sent to Great Britain for metallurgical examination and the preparation of macro-section photographs.

The greatest number of welders employed at one time was ninety.

Figs 49 (*c*)



Typical Welding Details

(The welding symbols used in these Figures are in accordance with American practice. A similar scheme of symbols is now incorporated in British Standard No. 499.)

Gravity Fenders

In addition to forming a special feature of the design allowing flexibility in the structure, the forty-four sets of three heavy gravity fenders produced some interesting erection problems. The fender cylinders arrived at site complete with trunnion and internal reinforcement, as shown in Figs 21, Plate 3, and *Fig. 50* (facing p. 287). In the yard they were sand blasted, primed, and coated with hot enamel before the three 14-inch-square Douglas Fir rubbing timbers were bolted in position. The fabricated trunnion hangers had the stainless-steel hanger pins held firmly in position in the 1⅜-inch-thick pin plates, which had been bored to a diameter of 6 inches +0·002 inch. The assemblies were completed in the yard by welding on the batten plates before cleaning, priming, coating and attaching the 8-inch-by-6-inch timber rubbing strips.

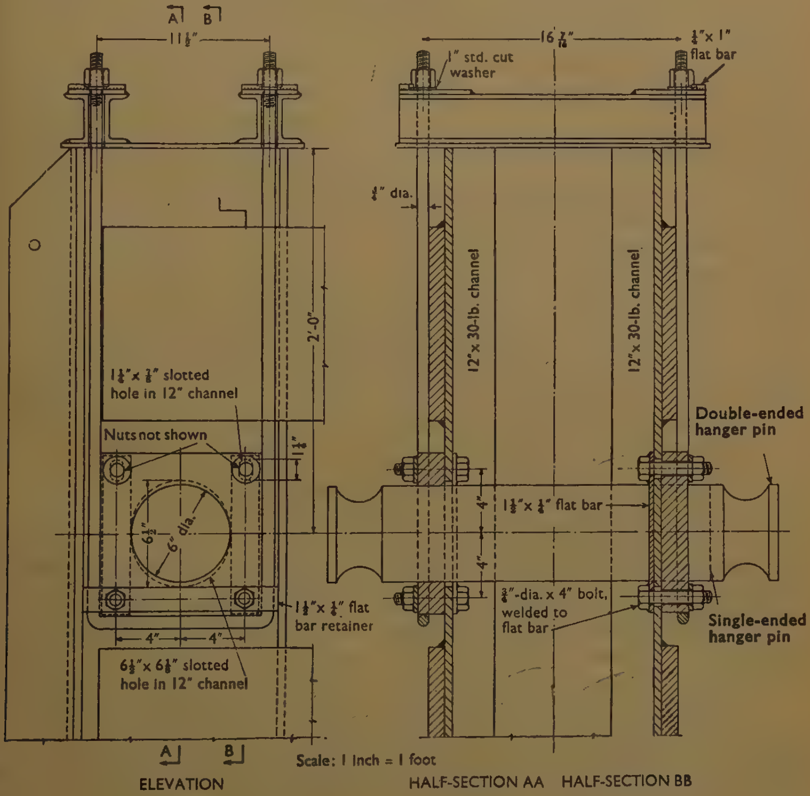
To ensure the maximum possible accuracy in pitching, each group of four fender piles was driven after the adjacent jetty structure had been framed. The next step in the erection was to cut off the fender piles, cap them and attach the cap beam and complete the members connecting the group to the jetty structure and weld on the guide beams. After this the trunnion-hanger assemblies were bolted in position at their upper ends ready for plumbing and aligning the pins. Figs 23, Plate 4, show how the batten plates permit the adjustment of the hanger pins in a plane at right angles to the jetty and also the longitudinal movement permissible at each pin in the plane parallel to the jetty face. Working from the middle pair of hangers it was, therefore, possible to line up and plumb the pins so that they were in the same vertical and horizontal planes. This work was carried out to quite close limits and without undue difficulty. After this lining up, the batten plates were welded to the fender piles and the hanger pins were welded to their pin plates. The group was now ready for emplacement of the empty cylinders by means of a 7-ton derrick crane mounted on a barge; the deck of the barge was sufficiently large to transport a set of three empty cylinders from the shore. The anchor-chain-steel suspension links, which were received at site in matched sets of four, were hung on the hanger pins and at low tide the crane barge lifted the empty cylinders into position (*Fig. 51*, facing p. 287).

The centres of the trunnions are located in the same vertical plane as the centres of gravity of the fenders complete with rubbing timbers, that is to say, 3 inches forward of the centre of the cylinders and therefore the latter, when empty, hung with their tops canted outwards. To prevent it being lifted off the trunnions with the rising tide each cylinder was ballasted with sea water. The final slight vertical adjustment of the top hanger pins was done at a subsequent low tide with the cylinders empty and their tops were pulled back so that they hung truly plumb. *Figs 52* show the method of effecting this, the bearing of all the links being carefully checked by feeler gauge. This adjustment ensured that the weight of the concrete which was the main load would be taken equally by the four links.

The filling of the cylinders was undertaken at another suitable low tide, the drain plugs having been removed and the bottom water pumped out before cleaning the inside and placing the concrete, which was brought by tipping lorries from a mixing plant located on the shore and deposited with the aid of chutes and baffles. Each cylinder required 23 cubic yards of 1 : 2·8 : 5 concrete made with a 2½-inch graded desert shingle, in order to obtain as dense a mix as possible, since the effectiveness of the fenders depended on their weight. Consolidation was effected by means of high-speed pneumatic vibrators and tests indicated a finished density of 154 lb. per cubic foot. Normal progress was one set of fenders per workable tide. After concreting, the cylinders hung vertically.

During rough weather which occurred in December 1949, two of the empty cylinders located at group No. 36 at the southeast corner of the cargo pier developed leaks and consequently lost their water ballast. They were then subjected to very rough buffeting by the waves but the storm did not last long enough to damage the links and the cylinders were lifted on to the deck, welded up at the bottom and replaced satisfactorily.

Figs 52



Arrangement for Final Adjustment of Gravity Fender Upper Links

Timber Decking

The laminated deck is uncommon in British practice. It forms a quick method of construction when plenty of timber is available. On this work, the sawn timber arrived already treated with Wolman salts (with a retention of not less than 0·35 lb. per cubic foot), and sawn to lengths to suit the stringer spacing and the break of joints. Six-inch nails at 18-inch centres were used for nailing each 6-inch-by-3-inch piece to the adjacent piece and galvanized iron clips were fixed with double-pointed nails at intervals of 2 feet to hold the deck to the stringers. A difficulty with this type of deck is to ensure that each strip fits tightly against the next and bears evenly on all the supports, a troublesome process unless the sawing is to close limits and all the timber is completely and uniformly seasoned. Jacks and wedges were utilized but it was not possible to obtain the desired tightness and subsequently, owing to shrinkage and road traffic, some "chatter" of the deck timbers on the stringers of the approach pier and cargo pier developed.

Cut ends and the bolt holes of the guard rail were brushed with a solution of Wolman salts at the time of erection, whilst all the bolts, which were a driving fit, had been dipped in coal-tar-base paint.

The fitting of the 4-inch-by-2-inch timbering on the oil pier was easier and this deck, which is not subject to traffic, has not yet shown any signs of loosening.

Installation of Pipework

All pipe lines are of mild steel. On the shore they are coated with bitumen and wrapped and laid below ground up to the anchorage block at the shore end of the pipeway. Near the anchorage block, insulated flange joints have been incorporated to prevent the leakage of cathodic-protection current. The block was completed up to the construction joint before the pipes were laid.

The 24-inch-diameter pipes for loading crude oil arrived at the site in lengths of 40 feet, with ends ready bevelled for welding. On the beach adjacent to the alignment, bearers were erected and pairs of pipes were joined together in accordance with the usual oilfield practice known as "roll welding." These pipes were then machine cleaned and lowered on to the alignment and end-welded on the lines already completed and resting on the roller supports previously welded on to the beams of the pipeway. By means of a cable attached to the shoreward end of each pipe it was gently pulled forward on its rollers. This lengthening and pulling forward was continued until the leading end of each pipe reached the junction of the approach pier with the oil pier—a distance of 4,000 feet. The pipes were then lifted locally to enable the bearer plates to be welded at the location of each roller bearing and at the same time the anchorage block was completed. Then the Z-type expansion bends were completed piecemeal and the pipes along the middle of the oil pier, together with the vertical expansion bends shown in *Fig. 53* (facing p. 296) were welded in situ.

The joints of the smaller pipes on the approach and oil piers were welded on the deck and the pipe was lifted into position. On completion of the welding of the pipes and the valves they were hydraulically tested before being brought into use.

All pipes received a base coat of red lead and then two coats of aluminium paint.

INSTALLATION OF JETTY EQUIPMENT

The biggest item of equipment was the heavy-lift derrick crane on the cargo pier. The clusters of foundation piles for this were driven at the same time as the adjacent bearing piles and the crane was erected without difficulty and was subjected to a load test of 53·5 tons as shown in *Fig. 31*.

The four portal cranes on the cargo pier were unused surplus war stock and were originally provided with their own diesel D.C. generating sets located in a cabin immediately above the portal. At Mina al-Ahmadi these generators were omitted and mercury-arc rectifiers, housed in a special building at the northern end of the pier, converted the 3,300-volt 50-cycle 3-phase AC supply to 440 volts D.C. Reels and trailing cables were fitted to the cranes to enable them to be operated at any position on the track. Erection of the crane structures was carried out with the aid of long-jib mobile cranes. Before commissioning, each crane was tested through all motions with a load of 7·5 tons. Their original war-time rating had been 6 tons at 60 feet but this was reduced to 5 tons.

The installation of the air compressors, six pairs of oil-hose derricks, air-driven salvage pumps, four 100-horse-power borehole fire-pumps, and air-conditioning equipment for the control and cargo offices was carried out without difficulty, as also was the standard-lighting system on the approach pier and oil pier and the floodlighting of the cargo pier. The motors installed had a total horse-power of nearly 1,000.

The original intention was to provide 4-inch laminated timber fire-walls at intervals under the decking of the oil and cargo piers, but it was decided to use water drenchers instead. These consist of 4-inch-diameter pipes connected to the fire mains, supported at a distance of 4 feet below the deck and extending right across the pier. The upper third of each pipe has three rows of $\frac{1}{8}$-inch-diameter holes at 12-inch centres. The water, when turned on, issues from these holes in fine jets and the drench or wall of spray which results should assist in the localization of any under-deck fire which may occur.

For the use of the pilot and mooring launches attending ships, three sets of boat steps have been incorporated in the structure at convenient places on the outside and inside faces of the oil and cargo piers.

CONSTRUCTION PLANT

With a few exceptions, all the plant used on the work was American. The following were the principal items used :—

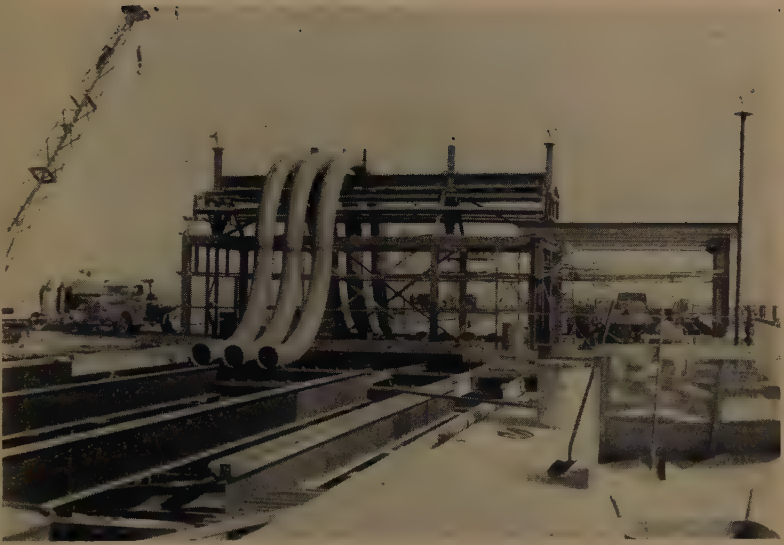
Sea-mule tugs	2
Landing craft medium 1 for fuel supply and service 2 for materials	3
Flat-topped bolted sectional barges 112 ft. by 27 ft. by 8 ft.	4
Floating pile-frame (described previously)	1
Erection barge with 7-ton derrick crane and 2 two-drum winches	1
Derrick cranes, 15 tons capacity	2
,, ,, 7 tons ,,	1
Divers' boats with equipment	2
Launch	1
Mobile cranes	5
Sand blasting equipment	8 sets
Shot blasting ,, (discontinued)	4 ,,
Winches or hoists, diesel-driven	2
Diesel-driven air compressors of 500 cubic feet capacity	10
Portable single welding sets, diesel- or petrol-driven, 300 or 400 ampere	55
Light and heavy motor transport	various
Bitumen-heating boilers	8
Generator sets 20–75 kw.	7
Portable lighting plants 5 kw.	10
Concrete-mixers	1

The contractor used high-pressure sight-feed acetylene generators with oxygen manufactured at the Kuwait Oil Company's plant for the flame cutting. Underwater cutting was done first by the usual oxy-hydrogen method and then by the oxy-electric arc process which proved to be very satisfactory. A well equipped workshop was provided for the maintenance of the mechanical equipment used.

LABOUR

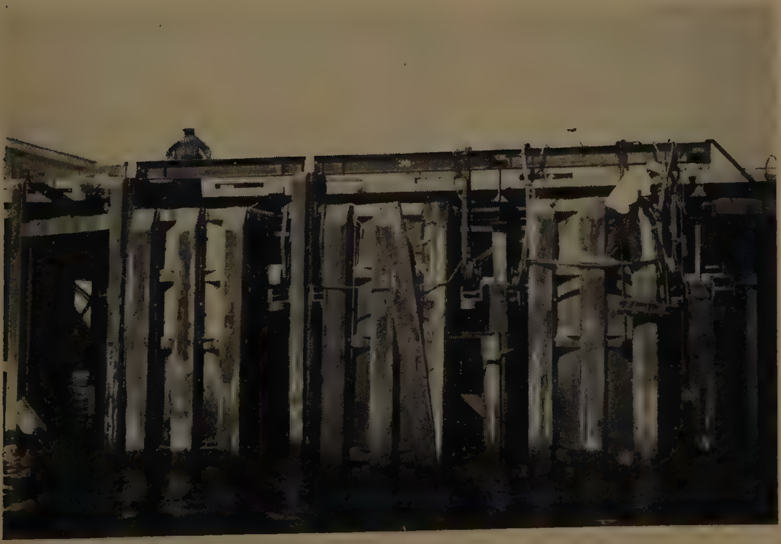
When work was at its highest a total of 300 Americans were employed on it, together with approximately 900 workmen from Kuwait, other Arab States, India, and Pakistan. All of these worked well alongside the American workmen. The site organization for all categories of labour included the provision of camps, necessary medical and health services,

Fig. 53



Vertical Expansion Bends

Fig. 54



Repairing a Damaged Fender

and the provision of suitable amenities. The construction period was free from epidemics or serious accidents.

QUANTITIES

The following quantities of the principal materials used illustrate the magnitude of the structure :—

H-section piles driven	3,853
H-section piling	11,400 tons (350,000 linear feet)
Structural steel	6,800 tons
Steel in gravity fenders and hangers	1,300 tons
Welding rods	151 tons
Coal-tar enamel	2,400 tons
Timber	4,300 tons
Pipe-lines	23 miles
Electric cable	23 miles
Magnesium anodes	75,000 lb.
Concrete	3,600 cubic yards

TIME OF CONSTRUCTION

On the 6th June, 1947, the contractor was given orders to undertake the work.

Driving of permanent piles was commenced on the 14th June, 1948, and was finished on the 9th October, 1949. The oil pier was commissioned on the 23rd November, 1949, when the S.S. Fort Bridger berthed and was immediately followed by two more tankers. The cargo pier was commissioned on the 6th January, 1950, on which day the S.S. Turkistan came alongside with a part-cargo of stores and equipment. The work had suffered delays owing to bad weather, the loss of a ship containing a consignment of piling and other materials, and various circumstances outside the control of the contractor ; nevertheless a period of 30 months from the beginning of the first construction drawing to the commissioning of the oil pier was very creditable.

THE COST OF THE STRUCTURE

The work was carried out on a fixed-fee basis and the approximate cost of the piers per square foot of decking and pipe-track areas, excluding the cost of pipe-lines and cathodic protection but including that of cranes, hose derricks, and other equipment, was £4 10*s*.

WORKING OF THE PIERS

The six oil-loading berths with their eight 24-inch crude-oil loading lines are fed by five gravity lines from the main crude-oil storage-tank farm at Ahmadi, about 6 miles distant, and by pumps drawing from a subsidiary tank farm at Mina.

The maximum hourly rate of loading so far recorded at a single berth, with no other tankers loading, has been approximately 4,000 tons per hour. The average hourly rate of delivery of oil per berth, with all six berths occupied, is approximately 1,600 tons per hour. The maximum tonnage hitherto loaded in 24 hours has been 172,000 tons, representing an average overall loading rate, inclusive of movements, of 1,194 tons per berth per hour.

The loading rates of furnace oil and marine diesel oil by means of the facilities provided at each oil berth are approximately 300 tons per hour maximum and 200 tons per hour average. The corresponding rates at the cargo berths are less.

Up to October 1951 the maximum number of tankers loaded at the oil pier in one month was 187. During the first 12 months of operation, loading was interrupted by bad weather for a total of 30 days, as compared with a figure of 76 days previously recorded at the buoy berths.

At the cargo pier, whenever possible, cargo is loaded directly from the ship into waiting lorries of capacities ranging from 10 to 15 tons. In spite of the 10-foot gap between the face of the gravity fenders and the edge of the jetty deck it has been found that modern ships are able to unload using their own derricks, so that in some cases both the quay cranes and ships' gear are used. The average rate of unloading for general cargo is approximately 40 tons per hour, using four cranes, and 60 tons per hour, using the ships derricks also at each hatch. Using the ships derricks alone it is considered to be possible to unload at a rate somewhat greater than with the cranes, but the former are able to load into a lorry only when the latter is placed at the extreme edge of the jetty whereas the cranes can load into vehicles banked several deep. The latter also possess considerable advantage in unloading pipes and structural members.

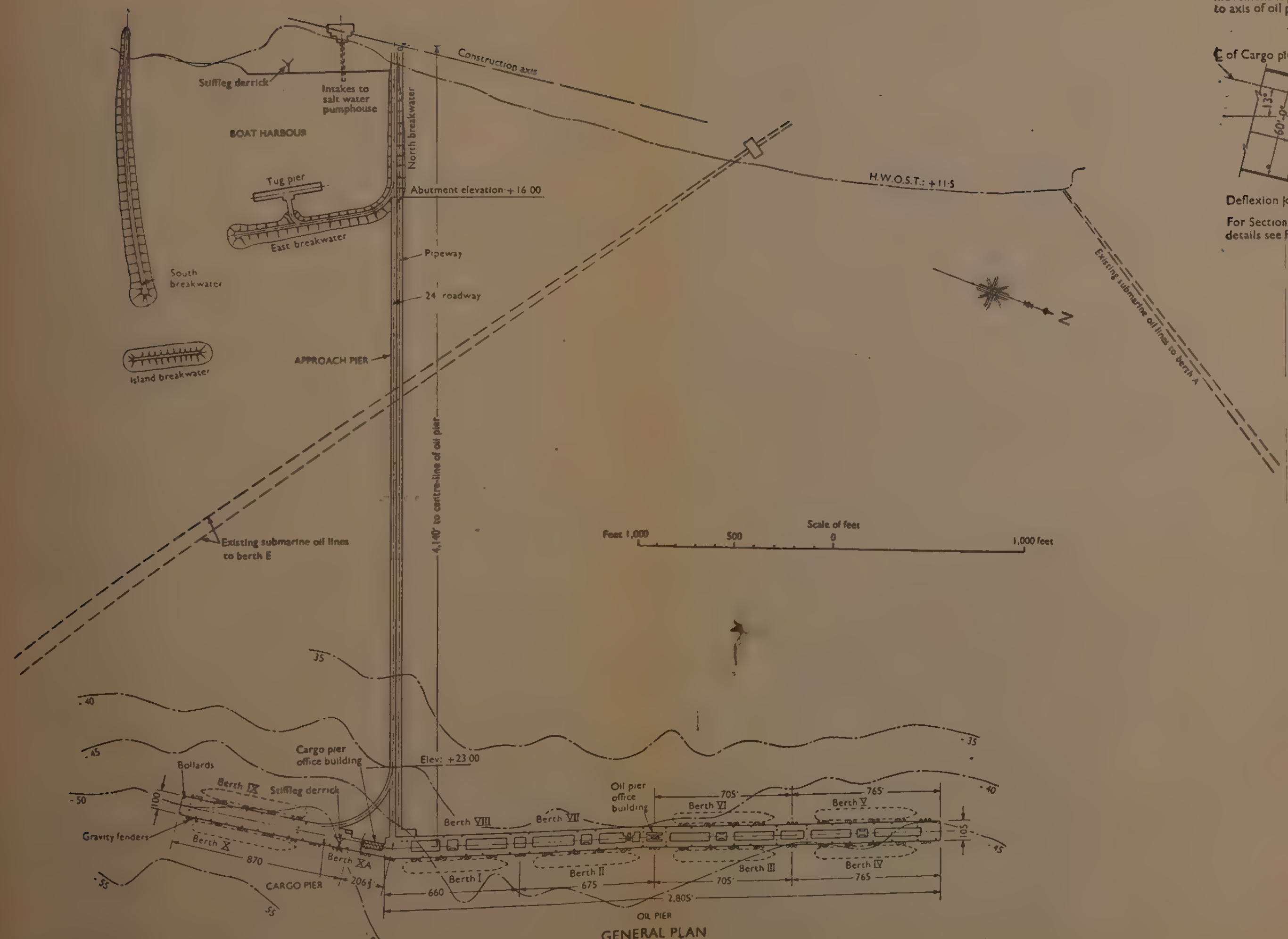
Initially, berthing was by daylight only, although the loading of oil and cargo handling was continued throughout the 24 hours. Subsequently, bulkhead light fittings were provided on the face of the cap beams above the gravity fenders, enabling ships to come alongside by night as well as by day.

MAINTENANCE

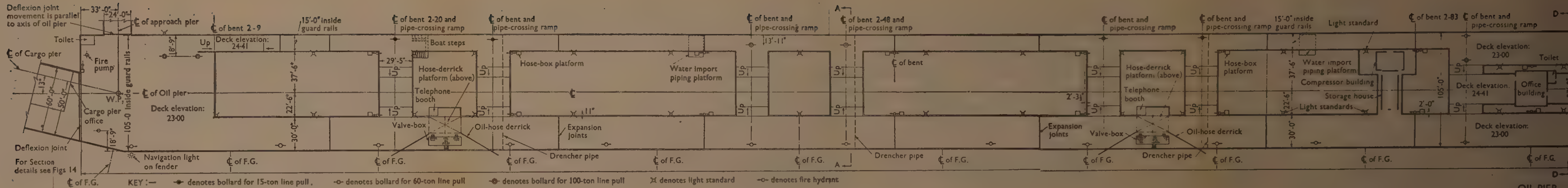
As already stated, there is every indication that cathodic protection will adequately provide for the protection of the structure below the level of +6·00. Between this level and deck-level, that is to say, in the upper tidal range and spray zone, it is anticipated that the effective life of the original two coats of coal-tar-base paint will not exceed 3 years. Systematic arrangements have therefore been put in hand for the maintenance painting of this portion of the structure on that basis.

OIL LOADING AND CARGO HANDLING FACILITIES AT MINA AL-AHMADI, PERSIAN GULF

FIG. 3



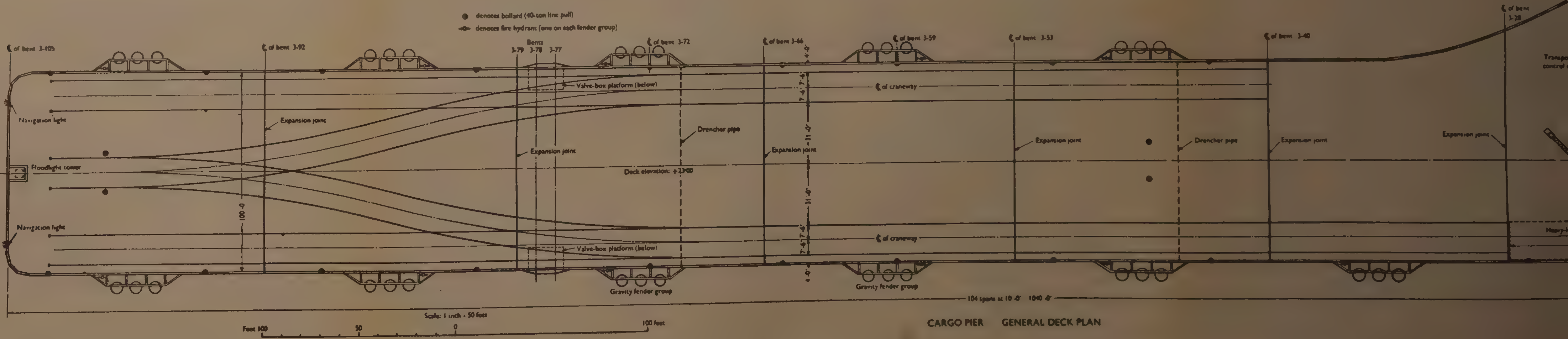
GENERAL PLAN



Deflexion joint movement is parallel to axis of oil pier
℄ of approach pier
Toilet
℄ of Cargo pier
Fire pump
Cargo pier office
Deflexion joint
For Section details see Figs 14
Navigation light on fender
℄ of Oil pier
Deck elevation: 23·00
℄ of bent 2-9
Deck elevation: 24·41
15'-0" inside guard rails
℄ of bent 2-20 and pipe-crossing ramp
Boat steps
Hose-derrick platform (above)
Telephone booth
Valve-box
Oil-hose derrick
Expansion joints
Drencher pipe
℄ of bent and pipe-crossing ramp
Hose-box platform
Water import piping platform
℄ of bent 2-48 and pipe-crossing ramp
℄ of bent
Compressor building
Storage house
Light standards
Light standard
Office building
Deck elevation: 23·00
℄ of F.G.
KEY:— denotes bollard for 15-ton line pull; denotes bollard for 60-ton line pull; denotes bollard for 100-ton line pull; denotes light standard; denotes fire hydrant

OIL PIER

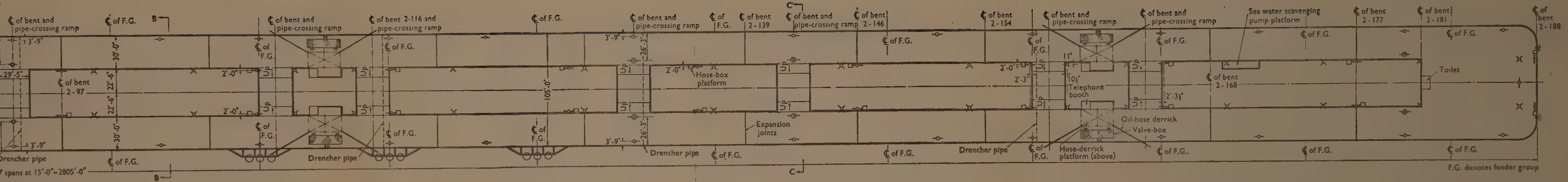
FIG. 6



CARGO PIER GENERAL DECK PLAN

William Clowes & Sons, Limited : London

PLATE I
OIL AND CARGO PIERS AT MINA AL-AHMADI



GENERAL DECK PLAN
1 inch = 100 feet

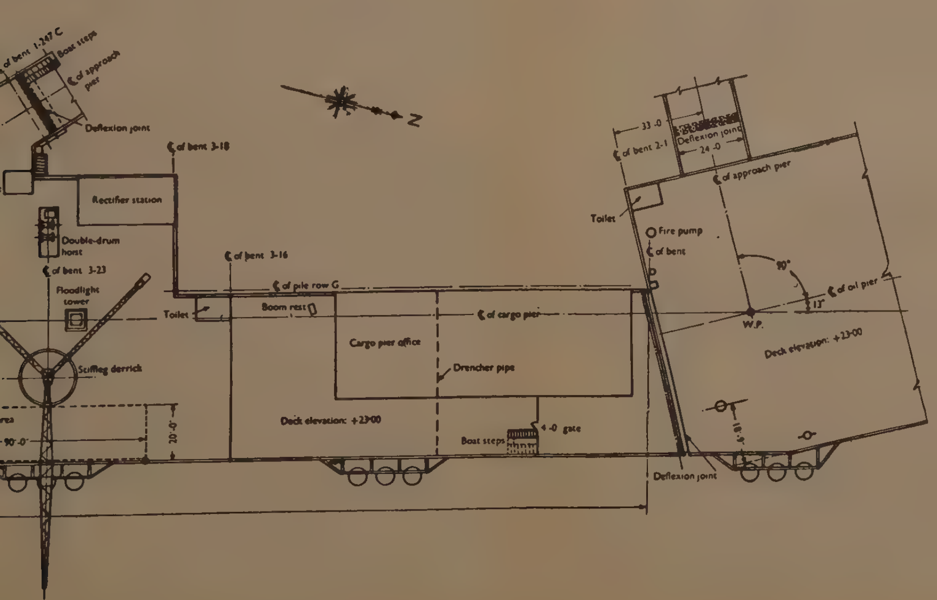
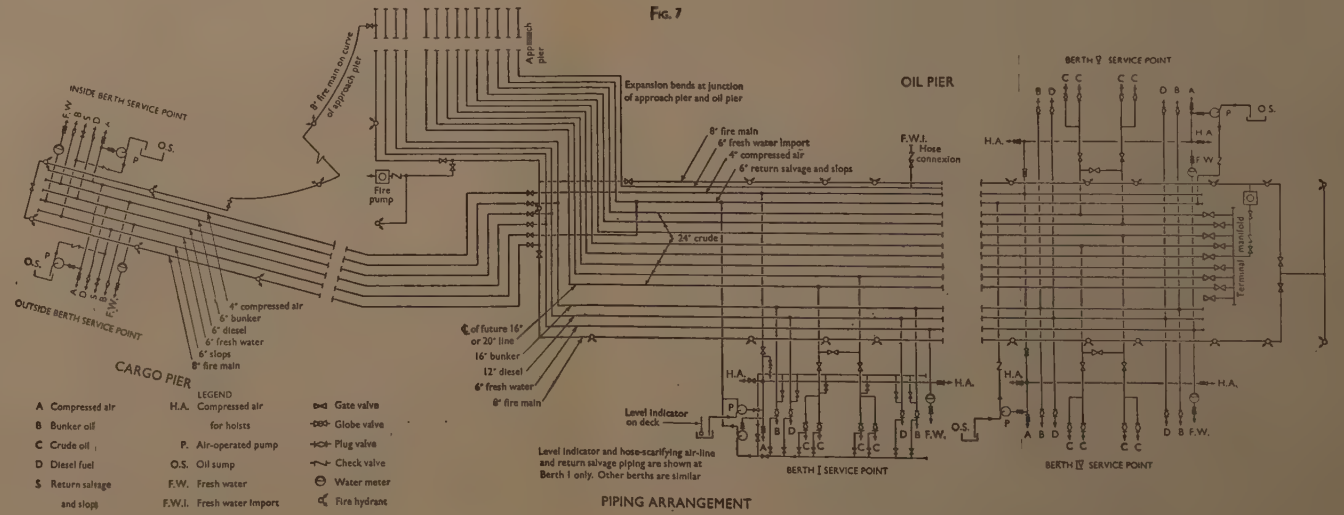


FIG. 7

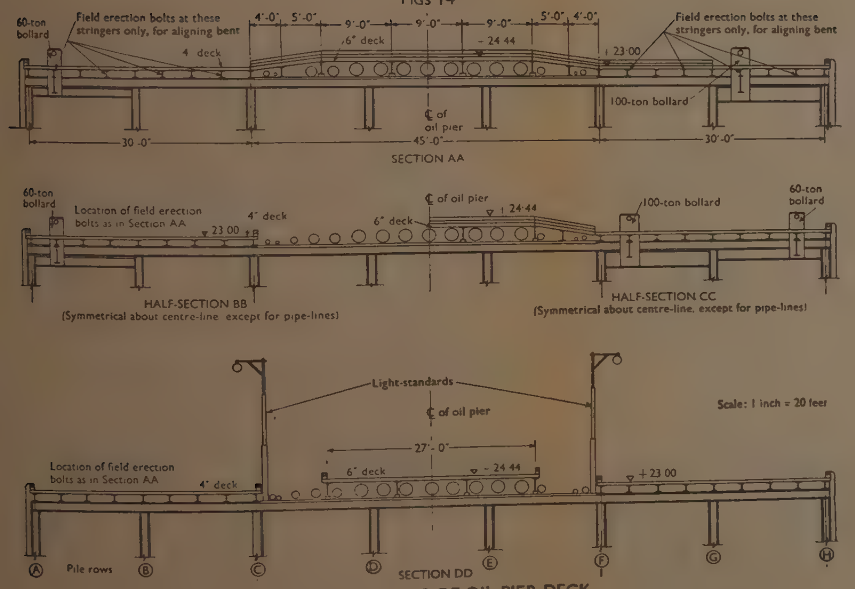


PIPING ARRANGEMENT

C. W. N. McGOWAN, R. C. HARVEY, and J. W. LOWDON

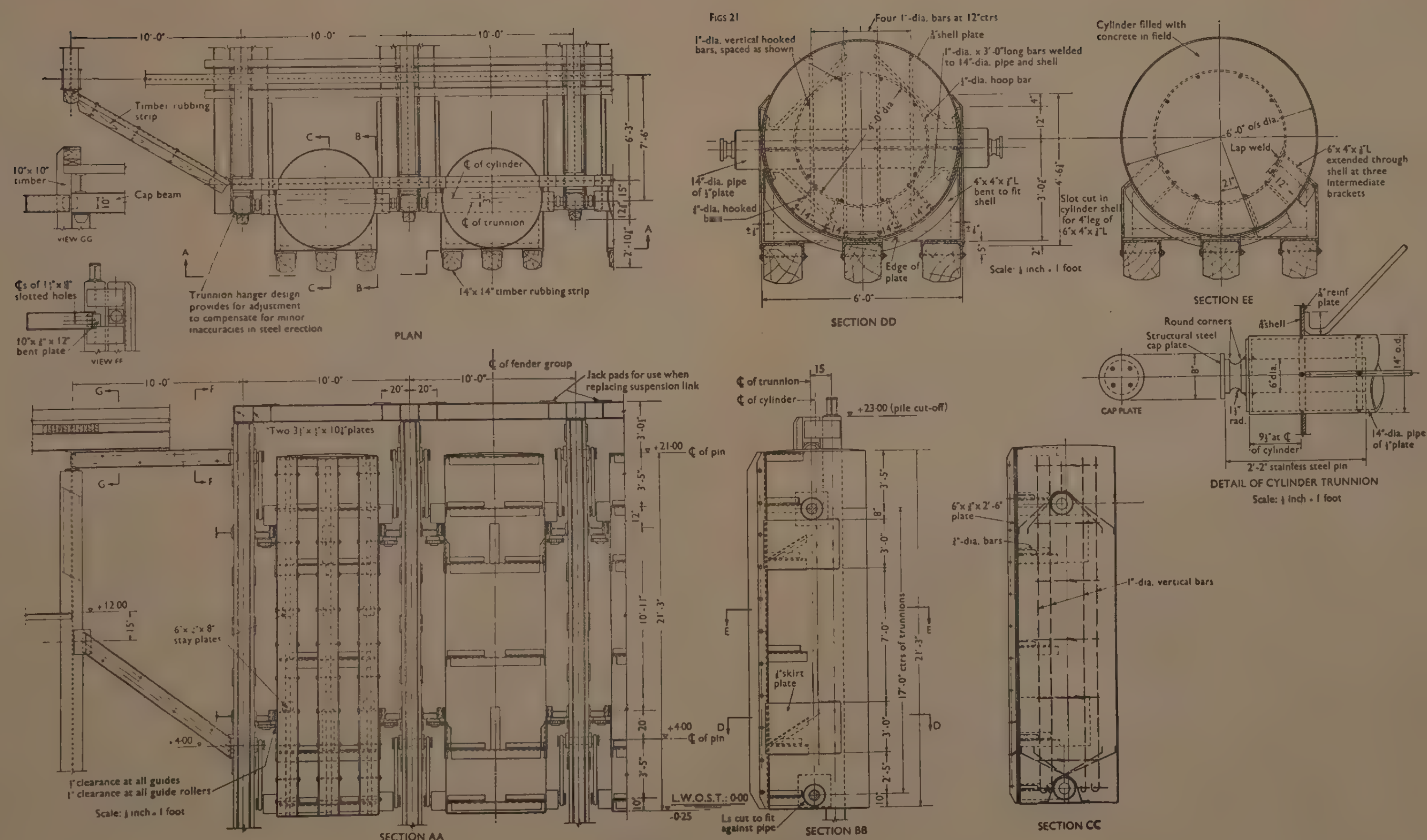
OIL LOADING AND CARGO HANDLING FACILITIES AT MINA AL-AHMADI, PERSIAN GULF

Figs 14



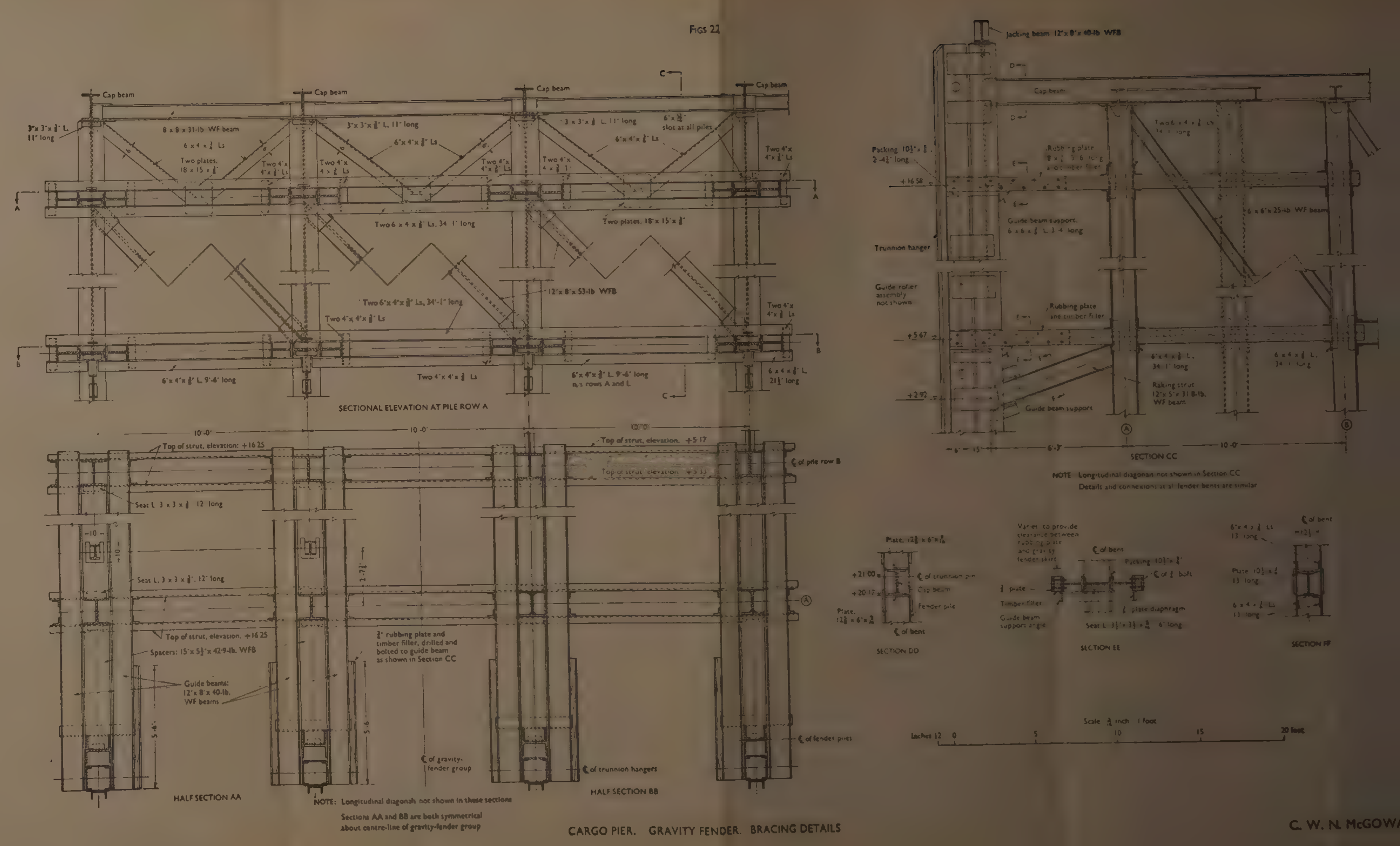
CROSS-SECTIONS OF OIL PIER DECK

Figs 21


10'-0" 10'-0" 10'-0"
Timber rubbing strip
10"×10" timber
Cap beam
VIEW GG
℄s of 1¼"×8" slotted holes
10"×⅝"×12" bent plate
VIEW FF
Trunnion hanger design provides for adjustment to compensate for minor inaccuracies in steel erection
℄ of cylinder
℄ of trunnion
14"×14" timber rubbing strip
PLAN
℄ of fender group
Jack pads for use when replacing suspension link
Two 3½"×½"×10½" plates
6"×½"×8" stay plates
1" clearance at all guides
1" clearance at all guide rollers
Scale: ¼ inch = 1 foot
L.W.O.S.T.: 0·00
Ls cut to fit against pipe
SECTION AA
SECTION BB
SECTION CC
℄ of trunnion
℄ of cylinder
+23·00 (pile cut-off)
+21·00 ℄ of pin
+4·00 ℄ of pin
17'-0" ctrs of trunnions
½" skirt plate
6"×½"×2'-6" plate
¾"-dia. bars
1"-dia. vertical bars
Four 1"-dia. bars at 12" ctrs
1"-dia. vertical hooked bars, spaced as shown
⅜" shell plate
1"-dia. × 3'-0" long bars welded to 14"-dia. pipe and shell
¾"-dia. hoop bar
14"-dia. pipe of ½" plate
¾"-dia. hooked bars
4"×4"×⅜" L bent to fit shell
Slot cut in cylinder shell for 4" leg of 6"×4"×⅜" L
Edge of plate
6'-0"
SECTION DD
Scale: ¼ inch = 1 foot
Cylinder filled with concrete in field
6'-0" o/s dia.
Lap weld
6"×4"×⅜" L extended through shell at three intermediate brackets
SECTION EE
Round corners
Structural steel cap plate
⅜" shell
⅜" reinf plate
CAP PLATE
1½" rad.
9½" at ℄ of cylinder
2'-2" stainless steel pin
14"-dia. pipe of ½" plate
DETAIL OF CYLINDER TRUNNION
Scale: ¼ inch = 1 foot

CARGO PIER—TYPE "B" GRAVITY FENDERS

Figs 22


Cap beam
3"×3"×⅜" L, 11" long
8×8×31-lb WF beam
6×4×⅜" Ls
Two plates, 18×15×⅜"
Two 4"×4"×⅜" Ls
6"×4"×⅜" Ls
6"×⅝" slot at all piles
Two 6×4×⅜" Ls, 34'-1" long
12"×8"×53-lb WFB
Two 4"×4"×⅜" Ls
6"×4"×⅜" L, 9'-6" long
6"×4"×⅜" L, 9'-6" long n/s rows A and L
6×4×⅜" L 21½" long
SECTIONAL ELEVATION AT PILE ROW A
10'-0" 10'-0"
Top of strut, elevation: +16·25
Top of strut, elevation: +5·17
Top of strut, elevation: +5·33
℄ of pile row B
Seat L 3×3×⅜", 12" long
Spacers: 15"×5½"×42·9-lb. WFB
Guide beams: 12"×8"×40-lb. WF beams
⅜" rubbing plate and timber filler, drilled and bolted to guide beam as shown in Section CC
℄ of gravity-fender group
℄ of trunnion hangers
℄ of fender piles
HALF SECTION AA
HALF SECTION BB
SECTION CC
SECTION DD
SECTION EE
Packing
Trunnion hanger
Guide roller assembly not shown
Cap beam
Fender pile
℄ of bent
Scale

NOTE: Longitudinal diagonals not shown in these sections

Sections AA and BB are both symmetrical about centre-line of gravity-fender group

CARGO PIER. GRAVITY FENDER. BRACING DETAILS

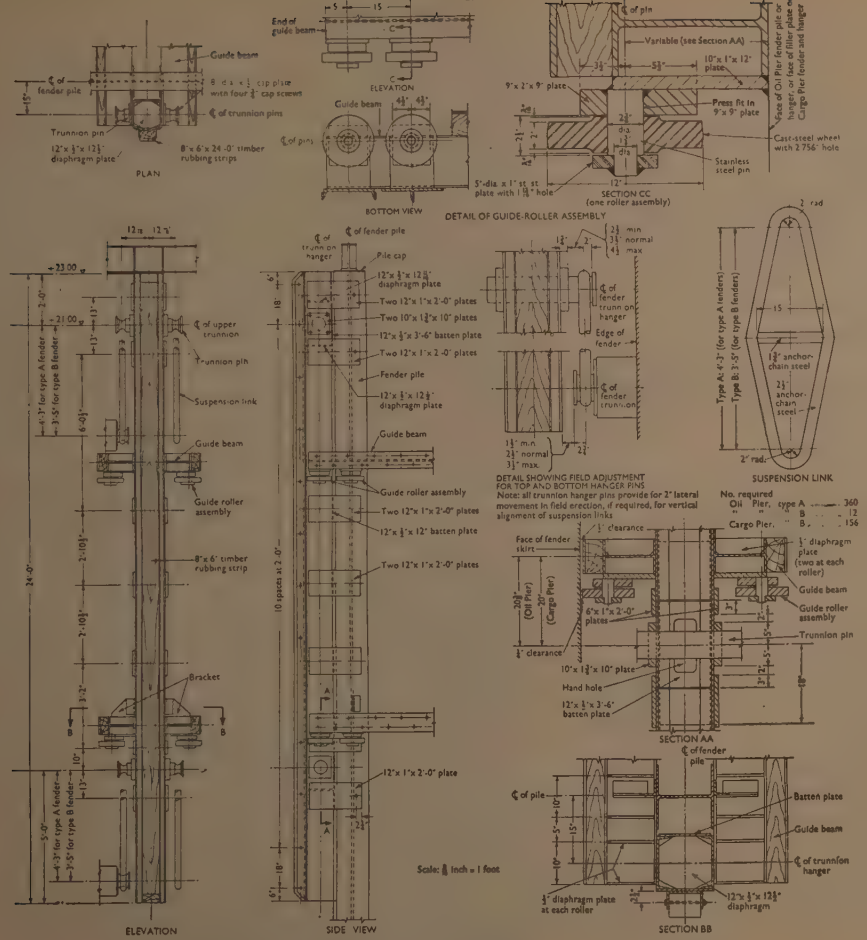
C. W. N. McGOWA

AM Clowes & Sons, Limited: London

PLATE 4
OIL AND CARGO PIERS AT MINA AL-AHMADI

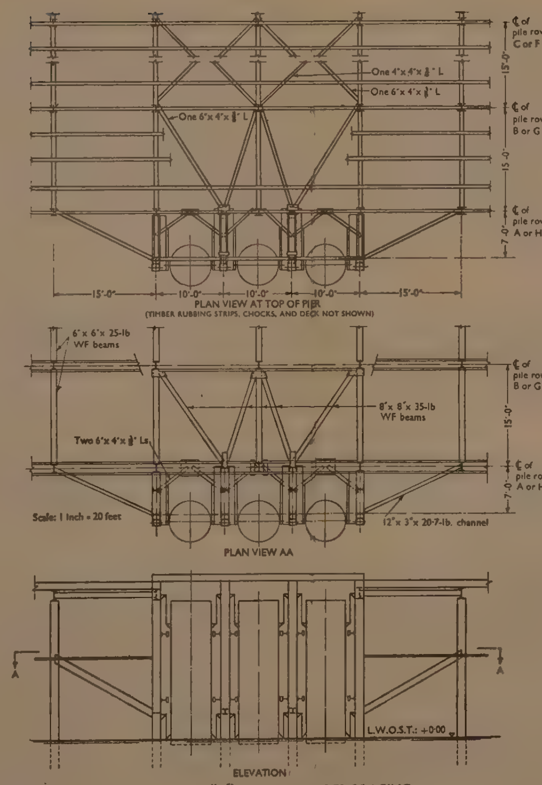
OIL LOADING AND CARGO HANDLING FACILITIES AT MINA AL-AHMADI, PERSIAN GULF

Figs 23



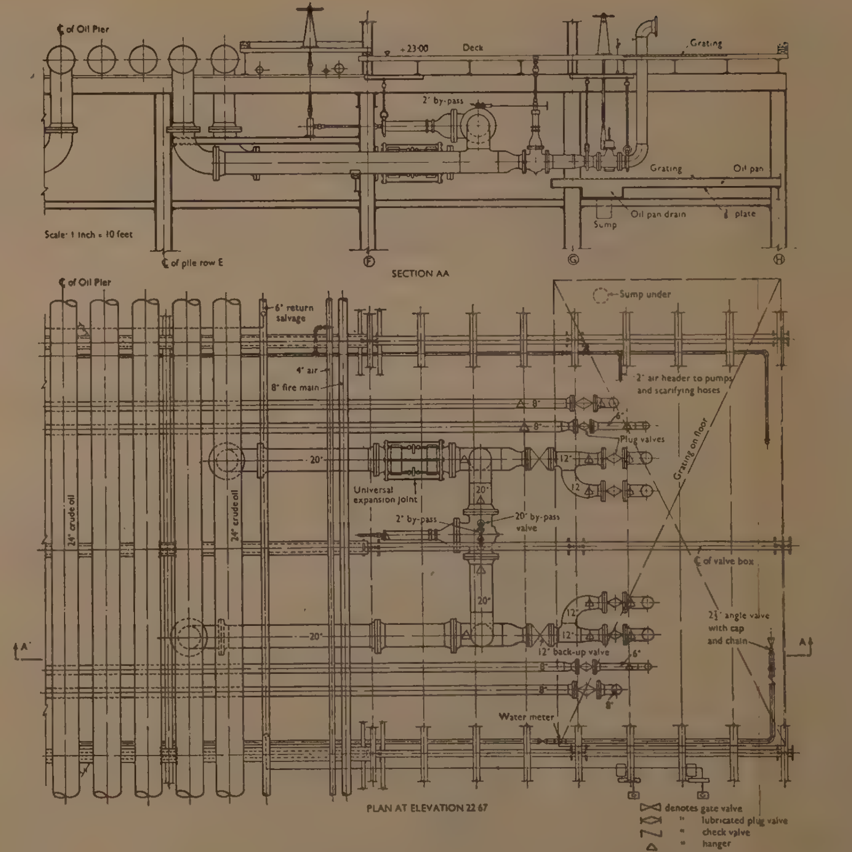
TRUNNION HANGERS, GUIDE ROLLERS AND SUSPENSION LINKS FOR GRAVITY FENDERS

Figs 24



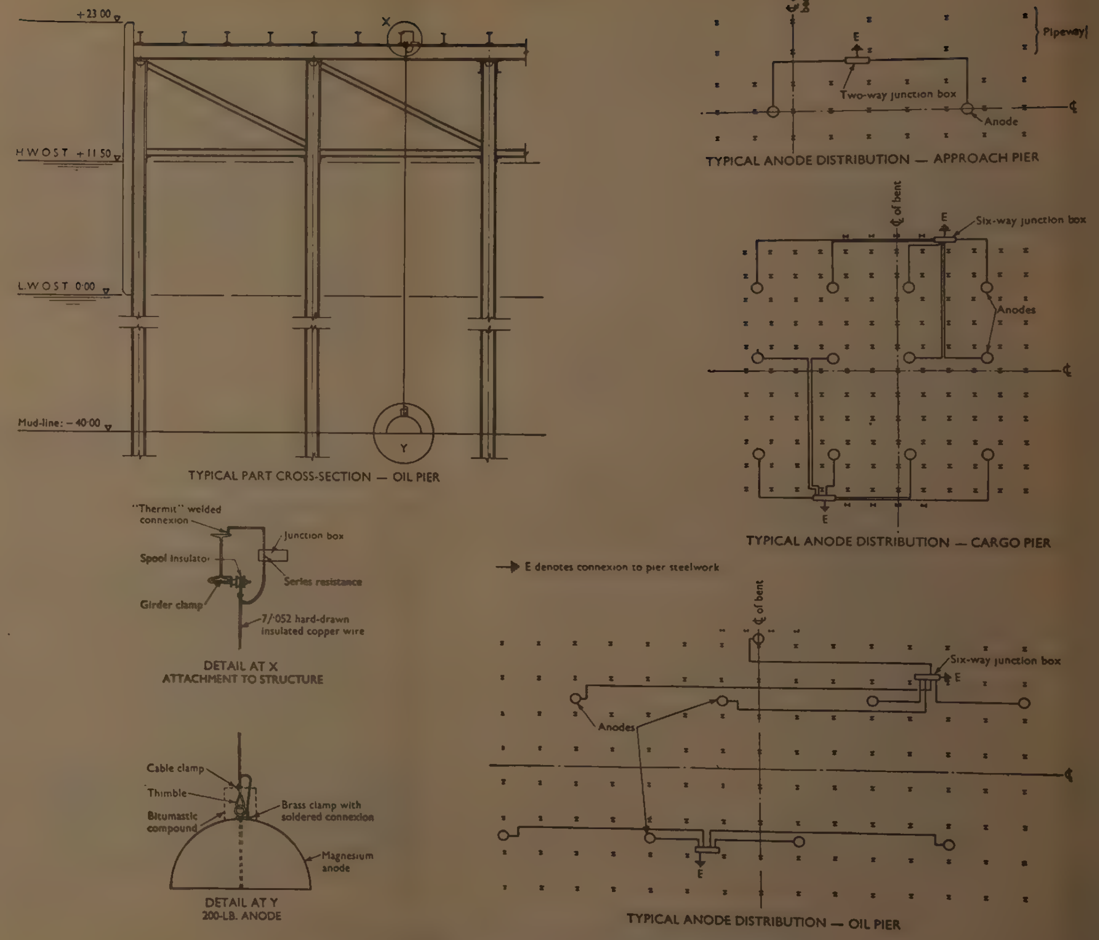
OIL PIER — TYPE "A" GRAVITY FENDER BRACING

Figs 29



SERVICE PIPING FOR OIL — LOADING BERTHS

Figs 41


TYPICAL PART CROSS-SECTION — OIL PIER
H.W.O.S.T. + 11.50
L.W.O.S.T. 0.00
Mud-line: − 40.00
DETAIL AT X
ATTACHMENT TO STRUCTURE
DETAIL AT Y
200-LB. ANODE
Magnesium anode
TYPICAL ANODE DISTRIBUTION — APPROACH PIER
TYPICAL ANODE DISTRIBUTION — CARGO PIER
TYPICAL ANODE DISTRIBUTION — OIL PIER
Two-way junction box
Six-way junction box
Anodes
E denotes connexion to pier steelwork

ARRANGEMENT OF ANODES FOR CATHODIC PROTECTION

The Institution of Civil Engineers Proceedings, Part II, June 1952

C. W. N. McGOWAN, R. C. HARVEY, and J. W. LOWDON

For the time being the treatment given consists of sand blasting and repainting with coal-tar-base paint of the original specification. At the same time, however, experiments are being carried out with a view to finding a more satisfactory coating. The tests embrace about fifty different treatments, including chlorinated rubber and bitumen coatings, and are being carried out in co-operation with the Sea Action Committee of the Institution and the Iron and Steel Research Association.

A point of interest in connexion with the corrosion of the structure in the spray zone is that the coating often appears to become defective initially at welds, the rusting then spreading beneath it. It is thought that this may result from attacks on the coating by traces of alkaline residue from the welding-rod flux.

Damage to gravity-fender rubbing timbers has been heavy, as might be expected where ship movements are so frequent, the rate of replacement being about eight per month. Damage to the jetty face rubbing timbers has been less, replacements averaging about three per month. No attack by marine borers has so far been observed.

Several cases of major damage to the gravity fenders themselves have occurred—usually when a ship has struck the extreme fender rubbing timber or the end trunnion-hanger rubbing timber in a direction at an inclination to the fender face-line. As a result, the fender supporting framework has been distorted longitudinally to the jetty, the distortion amounting in one instance to approximately 2 feet at the trunnion-hanger line and extending back to the third row of jetty piles. At the same time, hanger pins have been dislodged from their links, in one case a fender having been left suspended by two links only.

Some ingenuity has had to be exercised in the repair of these damages particularly since there is no crane available at the site capable of lifting a fender and opportunity for welding is limited to brief intervals between the berthing of tankers. *Fig. 54* (facing p. 296) illustrates such a repair in progress.

In spite of the maintenance difficulties mentioned above, this type of gravity fender has proved to be suitable for the conditions existing at Mina al-Ahmadi.

In future design it would be recommended that horizontal stiffening bracing should be added to prevent such longitudinal distortion in the event of collision of the kind described; that heavy timber fendering should be introduced between the end-trunnion-hanger rubbing timber and the adjoining jetty-face rubbing timber; and that the concrete hearting of the fender should be cast in sections to facilitate removal.

ACKNOWLEDGEMENTS

The Authors thank the Kuwait Oil Company, Ltd., for permission to present this Paper.

Representatives of the Anglo-Iranian Oil Company Ltd. and the Gulf

Oil Corporation of the U.S.A., the partners of the K.O.C., gave valuable help in connexion with the scheme.

Messrs Rendel, Palmer and Tritton were the Consultants for the project.

The Contractors were International Bechtel Incorporated of San Francisco, a group of American firms, of whom Messrs Sverdrup and Parcel prepared the detailed drawings under the direction of Mr D. C. Wolfe. Mr J. Clark Rankin was the Contractors' Project Manager at Kuwait throughout the construction.

Mr E. B. Ayres, of Atlanta, U.S.A., advised on the installation of cathodic protection.

The gravity fenders were of the type devised by Prof. A. L. L. Baker of the Imperial College of Science and Technology and were constructed under licence from Messrs Port and Harbour Fenders, Ltd, of London.

The steelwork was fabricated in the U.S.A. by the Bethlehem Pacific Coast Steel Corporation and the Western Pipe and Steel Company, who also manufactured the gravity fenders.

All derrick cranes were supplied by the American Hoist and Derrick Company and the portal cranes were made by Messrs Stothert and Pitt, Ltd, of Bath.

The coal-tar enamel and paint were supplied by the Wailes Dove—Hermiston Corporation of the U.S.A.

AUTHORS' RESPONSIBILITIES

Mr J. W. Lowdon was Chief Engineer of the Kuwait Oil Company at Kuwait; Mr C. W. N. McGowan was Chief Civil Engineer of the Company in London; Mr R. C. Harvey supervised the construction as Consultant's Resident Engineer at Kuwait.

The Paper is accompanied by twenty-one photographs and thirty-three sheets of drawings and diagrams, from some of which the half-tone page plates, folding Plates 1, 2, 3, and 4, and the Figures in the text have been prepared.

Discussion

The Authors introduced the Paper with the aid of a series of lantern slides.

Mr John Palmer recounted the early history of the project, which had involved a certain amount of discussion. His firm had sent a partner out to Kuwait first of all in 1944 and, as a result of that visit, had advised that the best place to put oil-loading berths would be round the corner in Kuwait Bay. As would be seen from *Fig. 1* (p. 251) there was a sheltered bay there. The Oil Company, however, had boldly put their sea mooring berths straight out into the Persian Gulf.

At the beginning of January 1947, his firm had sent out an experienced harbour engineer to examine the pros and cons for a loading jetty. In June 1947, they had summed up the position as it then stood by preparing, in collaboration with a well-known British firm of civil engineering contractors, a Report called "The Marine Construction Programme."

Fig. 55 showed the lay-out for the oil pier as copied from that Report of June 1947. Six oil berths were planned to the south along the outside of the pier, with a possible future expansion of two or more oil berths to the north. At that time there were no berths proposed on the inside of the oil pier. It should be noticed that no cargo pier was shown in that lay-out plan, because it was thought that cargo could easily and economically be unloaded into lighters and then taken in for landing in the lighter harbour, which was called, in Fig. 3, Plate 1, the "boat harbour." In the programme of June 1947, his firm had given its opinion that those six oil berths could be built, including the approach and the lighter harbour, in 28 months. As stated in the Paper, the actual achievement of the Bechtel Group had been to complete eight berths in 30 months, which was faster than many had thought possible.

Mr Palmer then showed a slide of the oil pier as built, giving half-sections through an oil-loading point and through a fender group. The slide showed that the pipelines had been taken across to the loading face under the decking If they had been taken straight across to the loading face, the roadway at that point would have had to be ramped up over the top of the pipelines, and that would have increased the height of the derrick portals.

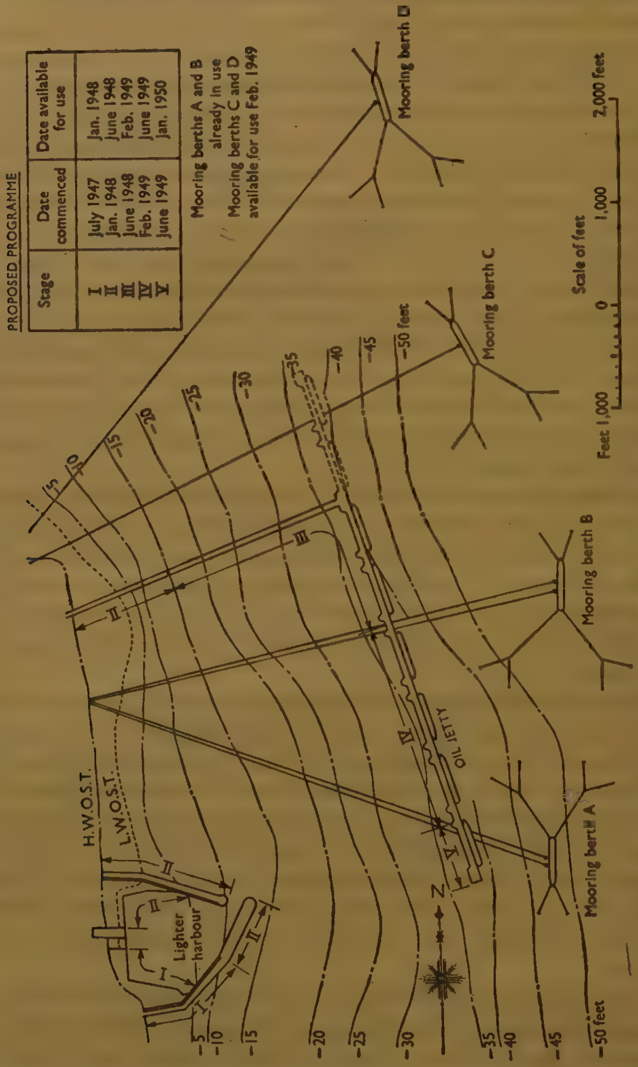
At about the same time as his firm had been working on their scheme, General Sverdrup had visited the site and prepared the Sverdrup and Parcel scheme. (Messrs Sverdrup and Parcel, referred to in the Paper as one of the firms in the Bechtel Group, were in fact better known in the United States as independent consulting engineers.) It was General Sverdrup who had produced the scheme for building a cargo pier, with which, so far as Mr Palmer knew, everybody was very pleased. He understood that it had done much useful work and he believed that at the present time it was proving a godsend in that it could also be used for loading oil.

The other modification which had been produced by Sverdrup was to have inside berthing for the oil tankers. The result of that, of course, was to shorten the length of the oil pier, but the approach pier in the Rendels lay-out shown in *Fig. 55* was only 3,000 feet, whereas actually the approach arm as built had to be a little more than 4,000 feet long in order to give ample water space inside the oil pier for the tankers to manoeuvre.

So much for past history.

There were one or two points regarding the design which might not have been readily appreciated. It would be noted that it was estimated that the oil pier when built would be unusable for perhaps 20 days of the year, whereas it had been forecast that buoy mooring berths might not be usable for 60 days in the year owing to rough weather. This difference was due, he believed, to the procedure adopted—namely, that the small mooring launch, when making a tanker fast to buoy moorings, had to get right alongside the buoy before the mooring wire could be passed through

Fig 55



PROPOSED PROGRAMME

Stage	Date commenced	Date available for use
I	July 1947	Jan. 1948
II	Jan. 1948	June 1948
III	June 1948	Feb. 1949
IV	Feb. 1949	June 1949
V	June 1949	Jan. 1950

Mooring berths A and B already in use
Mooring berths C and D available for use Feb. 1949

PROPOSED LAY-OUT OF OIL LOADING FACILITIES COPIED FROM REPORT OF JUNE 1947 the buoy, and in rough weather that was probably an impossible operation.

The 333-foot-radius curve in the approach to the cargo pier was, of course, introduced to allow for a future railway.

The northernmost oil berth was shown on the lay-out as 765 feet long, whereas the others averaged 675 feet or 700 feet. That extra length at the end of the pier had been wanted for two reasons: first, because it was necessary to place a bollard for head or stern lines the proper distance away from the tankers; and secondly, so that the northernmost fender group distribute any blow from impact of a berthing ship in both directions, northwards and southwards, through the horizontal truss to the piles.

The ratio of American imported labour to local labour—300 to 900—was a very high ratio for work in that part of the world; the reason had undoubtedly been the desire to produce a quick job.

Mr Lowdon had made some very useful suggestions for strengthening the fender-supporting framework. Mr Palmer thought that the Paper was a little lukewarm in describing the efficiency of the fenders—it stated that they had "proved to be suitable for the conditions." (Perhaps no more need be said.) He wondered, however, whether any other design of fender would have stood up to the heavy work of the past 12 months.

Mr Palmer then said he would venture on to the dangerous ground of economics and try to give the broad economic picture of this unusually big job. Using the figure given in the Paper of the cost per square foot and measuring up (from the drawings in the Paper) the area of pier, it could easily be computed that the oil pier, plus a proper proportion of the approach pier, had cost somewhere in the region of £2 million, which was almost exactly £325,000 for one oil berth. He had no knowledge of the cost of a buoy-mooring berth, but his estimate would be somewhere in the region of one-tenth or perhaps one-seventh of that amount. He wanted to draw attention to the advantages of the pier berths compared with ordinary buoy-mooring berths. One advantage was that it was possible to load oil on 340 days of the year instead of 300. Another advantage (which, perhaps, the Authors would confirm) was that it was possible to load oil faster through one 24-inch line than through two 12-inch lines. He believed that the considerably greater cost of the oil pier compared with buoy moorings must have been amply justified.

Mr Palmer also wished included in the acknowledgements the name of Mr Philip Southwell, Managing Director of the Kuwait Oil Company, who was really the father of the whole project.

Mr P. R. Robinson referred to the statement in the Paper that four uncoated test piles had been driven on the 13th September and pulled between 21st and 31st December, that was to say, after a minimum of 14 weeks. The Authors had stated that that period was too short for the final seize-up of the piles to take place. Of course, there might be local ground conditions which could cause such a long period to elapse before the final seize-up, but Mr Robinson was of the opinion that by far the greater amount of the tightening up of the ground round the piles would have taken place over a much shorter period, probably in the region of a month or so, and that any increase that would take place after a period of 14 weeks would be negligible. The frictional resistance of the piles as shown by the jacking tests would therefore be approximately the final resistance.

The rate of pile driving appeared to have been about one per hour in good weather, which was good—if perhaps not exceptional—for floating plant and the type of pile used; but the maximum number of forty which had been driven in 1 day was indeed a remarkable achievement, considering that a pile had to be pitched and driven, the rig hauled back, another pile picked up, and the rig moved forward again into position, all in 20 minutes, and all by hauling and slackening on anchor cables, even though the tidal flow was quite low—only ½ to 1½ knot. Presumably that rate had only been achieved in a flat calm. It would be interesting if a little more information could be given on the actual working of the winches and the manœuvring of the piling barge.

What tolerance had been allowed in the position of the piles both before and after driving? He asked because the Authors had stated that the longitudinal and diagonal bracings fitted remarkably well; he noticed, however, that that was apparently not only on account of the accuracy with which the piles had been pitched, but also on account of their flexibility, which would appear to indicate that some, at any rate, of the pile, had had to be forced into their correct position. If that was the case, could the Authors state whether any serious increase in stress had been caused thereby in the piles, since fixity was found to be about 5 feet below the sea bed?

On the question of barnacle attack, the Authors had stated that it was believed that proper application of the coal-tar enamel and the use of cathodic protection would prevent corrosion in the lower half of the tidal range and that subsequent examination of the structure had confirmed that. They had said, however, that the coating had been affected by barnacle attack after 1 year, and that it was intended to renew the coating every 3 years. The effect of the coating in the barnacle range therefore seemed to be to reduce for 2 or 3 years the current required for cathodic protection. The corollary of that was that the coating by itself was not a permanent protection against barnacles, which was what one would expect, because the attack of barnacles on coatings of that description was largely mechanical.

The laminated timber deck which had been used throughout was indeed uncommon in British practice. He had considerable doubts as to its suitability compared with concrete, more particularly with regard to fire hazard. That was not because the deck itself was particularly inflammable; indeed, a heavy timber construction was not at all easy to burn, even when not treated with Wolman Salts. He thought that an objectionable feature of the construction was that, in the case of the oil jetty, where spillage of oil on the deck was bound to occur, it could not be expected that the joints to the decking would remain tight even though it could be laid tight. That would be particularly the case where timber was seasoned in one part of the world and built into a structure in different climatic conditions; and in the case of Kuwait, with its very high summer temperatures, he would expect that inevitably the joints would open up; indeed, the Authors had said that it had not been possible even to lay the decking tight enough to prevent chattering. Therefore, he suggested that the danger of this form of construction was that the joints and the deck might become soaked with oil, which would constitute a fire hazard, whether the timber itself burned or not.

It would be interesting to know what comparison of cost had been made between a timber and a concrete deck and whether the timber had been adopted because it was a usual form of American construction, or whether there had been some other economic or technical reason. Naturally such a form of decking was quick to erect and presumably cheap when plenty of timber was available, but the Persian Gulf area was not remarkable for its large timber resources, and the decking all had to be shipped from the United States. Of course, a further point in relation to a timber deck was that it did allow a certain amount of movement to take place in the jetty structure, but so far as that was concerned the provision of joints in a concrete deck was not a matter of any great expense or difficulty. Therefore, since sand and shingle were local products of Kuwait, it seemed to him that, other things being equal, a concrete deck would have been more suitable to the case.

Professor A. L. L. Baker said that the Paper described what was, perhaps, the most important jetty structure in the world. He wished to make one or two comments in regard to the fenders and to ask one or two questions, the answers to which might be of value to port and harbour engineers.

The first point he wished to discuss was whether the arrangement of the fenders was most suitable in view of the site conditions. It appeared that occasionally tankers approached in such an oblique manner that the ship contacted the supporting framework of the fenders (*Fig. 56*). Provision for that degree of out-of-control berthing could only be adequately made by substituting the fender groups by rotating bell dolphins of the type used at Heysham. In view of the number of occasions in which oblique collisions had occurred in relation to the total number of ships berthed, which he believed amounted to nearly 200 a month, it was probably true to say that the higher cost of bell dolphins would not have been justified, and that the right solution to the problem was some additional longitudinal strengthening to the suspension frames which had already been proposed. The cost incurred in regard to the occasional collision was probably small in comparison with the cost of bell dolphins, which were certainly justified when fendering was liable to receive almost head-on collisions. Would the Authors explain the manner in which berthing was carried out so that Members could visualize under what circumstances oblique collisions might occur ?

In regard to blows normal to the jetty, a simple way of appreciating the significance of the fenders was to consider the increase of impact which would be received by the jetty structure if other forms of fendering had been used. The fenders could recede a distance of 4 feet. If the usual movement of only a few inches had been provided, the impact received by the jetty would have been about ten times as great, and since the horizontal forces were resisted by bending in the vertical steel piles, some idea of the additional piling costs which would have been entailed could be imagined.

Fig. 56

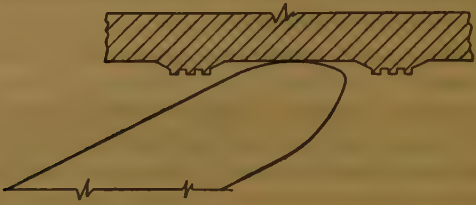
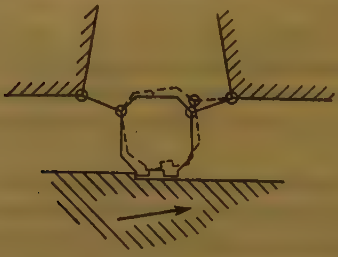


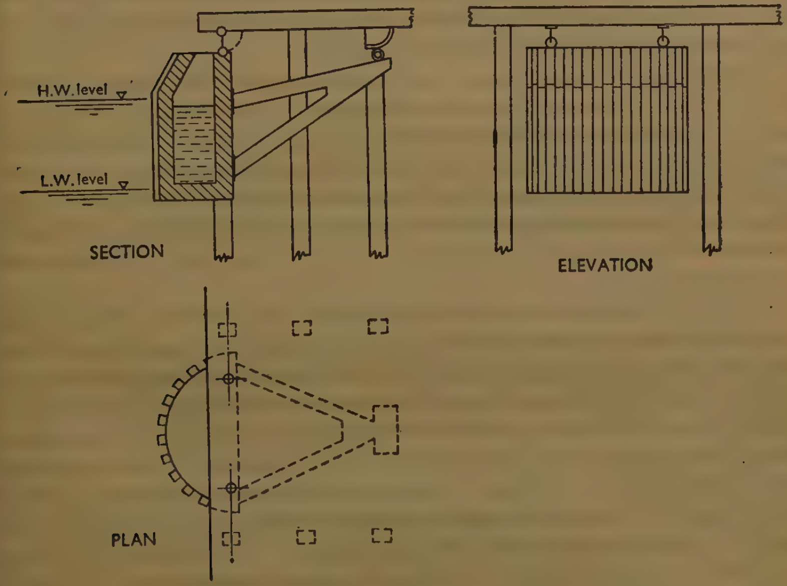
Fig. 57



PLAN

The fenders could perhaps have been a little lighter had the links been set out in a similar manner to those on the Heysham jetty. That arrangement would have allowed for a greater movement of the fender, but the rubbing strips would have developed a slight batter in the extreme position. The Heysham fenders were ballasted with pre-cast concrete blocks for easy emoval in case of repairs being necessary. It was interesting to note that a similar arrangement had now been proposed for extension work at Kuwait. Increasing the width between the suspension points (*Fig. 57*) might have been of some advantage in allowing the fender greater freedom longitudinally and in rotation. That would not have eliminated the collisions with the framework supports, but might have reduced the number of occasions on which the rubbing strips had been torn off. Such an arrangement of the suspensions was usually only justified when severe longitudinal rubbing occurred under normal conditions of berthing. It could be concluded, therefore, that the arrangement of the fenders could not with any great advantage have been otherwise.

Figs 58



It might be of interest to consider the possible advantages of replacing groups of fenders in some future jetty by a small reinforced-concrete suspended caisson, as shown in *Figs 58*. The three-point suspension would provide resilience to blows received from oblique as well as normal directions. The need for low-water bracing would be eliminated, but the tension in the links would be greater, and longitudinal rubbing at low water not quite so securely resisted as when low-water brackets were provided. Damage might require removing the unit, but it could be floated in and out of position. It presented, of course, rather a large area to wave-slap.

Could the Authors supply answers to the following questions:—(1) How frequently had damage to the fender supports occurred in relation to the number of berthings? (2) What reduction in time of turn-round had been made by the use of the special fenders, and therefore what saving in shipping costs ? (3) How much had the fenders been observed to recede under various conditions of impact ? (A comprehensive answer to that question would provide much valuable information for the design of future marine structures.) (4) What up-and-down movement could occur under severe weather conditions when the tankers were tied up ?

Mr D. H. Little said that the jetty described was such a big one (representing more than most engineers tackled in a number of jetties in a lifetime) that one was inclined to be afraid of commenting on it.

The area of the jetty amounted to 500,000 square feet, which was very big, and at £4 10*s.* per square foot made the cost about £2,250,000. Why had it been necessary to make the main berths as wide as 100 feet ? If they could have been 50 feet wide and the approach 15 feet wide, the area could have been reduced to 300,000 square feet.

Without good maintenance, his experience was that heavily braced steel jetties had a useful life of about 20 years. The bracing as a rule was relatively light sections. Water seemed to hang around brackets and on flanges of channels and beams, and so accelerated deterioration. He suggested that good maintenance would consist of painting every 3 years, with a squad of men doing one-third every year, and that, he imagined, would cost about £50,000 a year, which added another £1 million on to the capital cost of the jetty.

A solid reinforced-concrete slab for the deck, only 18 inches thick, would have eliminated all normal bracing, and he thought it would have vastly improved the maintenance problem. Usually with a plain pile without attachments to it, the water seemed to run off and the jetty to last longer ; painting was also much easier when there was no bracing. Such a deck would have needed only 25,000 cubic yards of concrete. It had been suggested that there was plenty of aggregate at Kuwait. He had assumed that it had not been used because it was not suitable ; but he was sure that it would have paid even to import aggregate.

With regard to fenders, the weight of steel used was given as 1,300 tons, and at a rate of £100 a ton for steel alone, the total cost of fendering must have been £200,000—that was to say, 10 per cent of the whole cost of the jetty. This would be very low for a normal jetty but it was not a normal jetty at Mina. The area in relation to the length was very large—twice the normal area for such a length of jetty. Even for a jetty of normal area, the cost of fendering (which would then be 20 per cent of the total) was still relatively cheap. He thought it could have been made cheaper still by providing less fendering. He suggested that there could have been three groups of two clumps for each berth instead of four groups of three clumps. That would have halved the amount spent on fendering.

But more important was the type of fender. Professor Baker had designed fenders of that sort for the Admiralty during the war, and had himself christened the swinging clumps " dead men " and the frames in which they swung " coffins." By the time one had got a dead man into a coffin, there was only one more thing to do with him—and he thought that the present meeting constituted the funeral service of those fenders!

It would be noted that the fenders were swung vertically. Mr Little had always conceded that they probably had a certain amount of longitudinal resistance, but the bottom support necessitated so much bracing at low water in the jetty that he had never favoured them. He thought that on this particular job, if there had not been so much tidal bracing, it would have been possible to save £500,000 on the jetty and to halve the maintenance costs. He would say that the gross cost of the fendering in the jetty amounted in all, when everything was capitalized, to probably half the cost of the jetty, if one added the cost of maintenance and extra bracing.

With all jetties the main problem was the fendering, and he did not think it had been solved yet. It was a big disappointment to him to read in the Paper that the fenders had not behaved so well under longitudinal stress. From the plan it could be seen that the whole of the fenders stuck out and simply asked to be hit—and apparently they were being hit! The Admiralty had designed gravity fenders and used them on three different jetties in which, the concrete clumps were slung horizontally instead of vertically. That eliminated low-water bracing entirely, but it did not give the same longitudinal resilience as he had hoped the vertical ones would have. Vessels of up to 30,000 tons were using the jetties, although certainly not at the rate of two hundred per month; and so far the fenders had behaved quite satisfactorily. The first had been suspended from two supports, one at the back and one at the front; the other two had been suspended with four supports, two at the front and two at the back. That four-point support had not been satisfactory, and he had been interested to hear about Professor Baker's three-point suspension, because the Admiralty were thinking now of turning the fenders so as to lie flat, holding them with two supports at the front and one at the back. Whether anything could be done in that way, Mr Little did not know. He hoped they would be able to swing sidewise just as they did backwards and forwards. He did not pretend that they had solved the problem, for they had not; but equally he did not think that the Authors had shown a solution. The complicated bracing system made their jetty look like a seaside pier rather than a modern tanker berth, and it brought with it all the ultimate maintenance problems. He did not think they had the best answer; he did not know the answer yet, but he was certain that the best answer had not been given in the Paper.

Whilst it might be felt that the fenders on the Mina piers were not the best answer to the problem, it should be remembered that it was a very remarkable feat of modern maritime engineering that huge vessels of 40,000 tons could be brought alongside an open piled structure at all; particularly in such exposed waters. There were other open piled jetties being built for similar vessels at various sites in the British Isles and all of them included some form of flexible fendering device. Most of those fenders derived much of their inspiration from Professor Baker's examples and all of them owed their basic principles of really worth-while flexibility to him. Professor Baker's pioneering work on flexible fendering was one of the most outstanding of British contributions to maritime civil engineering and whilst criticism of development work might be permissible—indeed it was an essential ingredient of progressive development—Mr Little's remarks should not be taken as derogatory to the value and significance of the primary work.

Mr M. J. Tomlinson commented on the test boring referred to on p. 252. The decision to put down trial borings had been made after several preliminary test piles, which were steel tubes, had broken without making any penetration of the rock on the sea bed. The result of that disconcerting information was, as the Authors had stated, that instructions had been given for trial borings to be made on the site of the pier as a matter of urgency. The boring team and 7½ tons of equipment had been dispatched by air to Kuwait within 2 weeks of his firm being given the order to start.

The whole of the work had been completed in 3½ months, including the deep holes and shallower wash borings mentioned in the Paper, and also a number of borings in the small-boat harbour. A combination of chiselling, rotary drilling, and blasting had been needed to get through the rock. A total of about 10 lb. of explosive had been required, in three successive charges, to blast a hole through the 3–4 feet of conglomerate on the sea bed. Fairly calm water was needed for rotary drilling from floating craft. For most of the work the craft had been anchored nearly a mile off-shore in 50 to 60 feet of water, and had been exposed to gales from all directions. In all, 4 weeks work had been lost owing to gales and rough seas during the 8 weeks on the water. However, in spite of the gloomy prophecy of the Oil Company's marine department, that they would not even be able to keep the barge anchored in position, the borings had been finished 3 months before the main contractor had started driving the permanent piles.

It had been realized by the consulting engineers that the wash-boring methods used to penetrate the sandy material below the rock were not ideal for identification of the soil, but it had given the main contractor all the information he needed for the piling work and had also enabled the job to be finished in quick time.

The boring platform had been cantilevered out from the side of the barge. Heavy steel decking had been necessary to prevent the borehole casing from buffeting and lifting the decking in rough seas. A diver had been indispensable in order to locate the holes when the barge was blown by the gales off the hole position, and also to lay the explosive charges to get through the rock.

With regard to the protective coating to the steel piles, some measurements had been made on the stripping during driving of various forms of coating used on the experimental piles referred to on p. 277. Those piles had been driven at his firm's Southall depot through 25 feet of sand, clay and gravel. Afterwards they had been dug out for inspection. One form of rubber latex coating had been stripped from 80 per cent of its area, and another type of rubber latex had lost 16 per cent. Coal-tar-enamel coatings had been tested. The American enamel, which had actually been used on the job, had been stripped from 10 per cent of the area of penetration, and the same type of enamel of British manufacture had been stripped from only 6 per cent of the area.

There was no information given in the Paper on the construction of the small boat harbour. The borings had shown hard calcareous material resembling a soft rock over most of the harbour area. Could the Authors say what methods had been, or were being, used to dredge that material?

Mr K. A. Spencer remarked that he had had the signal honour of being one of the party who had fixed the first magnesium anode to the Kuwait jetty. Thereafter, he had naturally been closely interested in the cathodic protection. It was a rather novel feature, about which little seemed to be generally known, but which is finding increasing favour.

In the Kuwait installation, he believed that immediate polarization had not been effected and that the current output of the anodes had been fairly high—about 5½ to 6½ amperes to a 200-lb. anode. He believed that that had now been reduced to 2 amperes. Would the Authors confirm that, to facilitate the polarizing of the jetty and the build-up of the calcareous deposit, welding sets had been used. The question of using magnesium initially had been based on the fact that there was no cheap power available; but now that that was no longer the case, it would be very much more economical to use power on such an installation.

The economic aspects of the protection problem had not been dealt with in the Paper. Had the Authors any figures for the relative cost of the coating of the piles (so much of which had, in any case, been stripped off during driving and by barnacle action) compared with the cost of the cathodic protection which sealed up the holes in the coating? That did not imply that the piles should not be coated, for coating and cathodic protection are complementary.

Mr J. H. Jellett enquired what was the degree of accuracy which was necessary in the berthing of the tankers. He asked because it was apparent that tanker traffic required a very quick turn-round, and in that respect it was very similar to the traffic which had to be handled at Southampton Docks.

At Southampton the operating authorities were shortening the turn-round time so that all the mobility of the various mobile equipment that had been provided on the quay—such as cranes—was devoted to moving them into a certain position ready to receive the ship. It appeared that a period of even 5 minutes used in moving the cranes to their correct positions after the ship had berthed was a source of annoyance to the operating authorities. So it was becoming more and more necessary that ships, even of the size of the "Queen Mary" and the "Queen Elizabeth," should be spot-berthed to within a matter of a couple of feet rather than that they should face the necessity of moving about seven cranes, a couple of conveyors, and half a dozen gangways a few feet into another position. Did similar conditions prevail with tanker traffic?

From remarks which had been made in the discussion, there was no doubt that the question of fendering was still a great source of debate, not only in regard to the actual details of fenders, but in regard to the conditions for which the engineer should design. For instance, at Kuwait, allowance had been made for an impact velocity of 0·75 knot, but it appeared that, whatever velocity one allowed for, sooner or later that velocity would be exceeded and damage would be caused. Professor Baker had shown in *Fig. 55* the position in which a ship might cause damage by striking the fenders at the incorrect angle and had referred to such conditions as the ship being out of control. It was, of course, a well-known fact that ships did occasionally get out of control and expensive damage occurred. Even in such a very sheltered port as Southampton, a large passenger liner had got out of control quite recently and she had taken a 6-ton quayside crane into the water with her before complete control had been re-established. Such things did happen, and it seemed to Mr. Jellett that the risk of such happenings called for a conference between engineers and marine underwriters, to find out what was the underwriters' idea of catastrophic conditions; because if engineers could so design fenders that they would prevent damage happening except in conditions under which the underwriters would accept the cost of damage, then they might find themselves very popular with their employers.

Mr H. S. Waddington said that although his remarks about fendering would possibly appear to be flippant, they were intended to be taken seriously. In his opinion, engineers should take care not to make fenders too efficient. There was such a large amount of give in the fenders described in the Paper that it was an invitation to those in charge of ships to come in at a good pace. He thought that if the Kuwait jetty had been constructed so that it *appeared* to be a solid mass-concrete structure, there would not have been the same amount of damage to the fenders.

⁂ **Mr J. A. Williams** referred to the design figures given on p. 263, for impact effects on the gravity fenders. Taking the quoted design figure of 672,000 foot-lb. for the energy to be absorbed by a fender group, and the assumed approach speed of 0·75 knot, it would appear on checking back, that the impact effect of a tanker had been calculated on the dead weight (30,000 tons) of the vessel. Mr Williams thought that the calculation should have been based on the displacement tonnage, which, for the tankers in question, might be of the order of 40,000. He realized, of course,

---

⁂ This and the following contribution were submitted in writing.—SEC. I.C.E.

that the tonnage had far less effect on the energy generated than had the velocity—especially when the latter was more than 1 foot per second—but, in a calculation which was in any case extremely arbitrary, it would seem desirable to introduce as many positively known factors as possible.

It had been mentioned during the meeting that measurements had been made of the actual fender deflexions when vessels were berthing, the speed of approach being measured simultaneously by stopwatch. Could those test results be made available ?

Mr D. A. Brown hoped that in the case of the pulling tests on the shore piles, the Authors would give some details of the incremental rises of the piles with the load.

With regard to the deflexion test, the right-hand graph in *Fig. 37* (*b*) (p. 276) had been plotted as two separate lines. It appeared to him that it should have been drawn as a loop similar to a hysteresis curve. He did not know what relationship the maximum transverse load in the test bore to the working load in the jetty, but it was evident that the top of the pile had acquired a permanent displacement of about an inch.

Had any succeeding tests been done in order to see if the permanent displacement continued with repeated loading ?

The Authors, in reply, thanked Mr John Palmer for the information which he had given regarding the early history of the project. Taking the pier out further into the sea had been a bold step, but it had been justified, and ships were berthing satisfactorily at the inner berths. Also a more compact and economical lay-out had resulted than if all berths had been on the outside face. The provision of separate cargo berths Nos IX and X (see Fig. 3, Plate 1) had been a definite advantage, since those two berths were arranged to be operated independently of, and without affecting, the oil loading at the remaining berths. Now, as Mr Palmer had mentioned, those berths were also useful for oil loading.

He had been correct in saying that one of the difficulties in operating buoy berths was the attachment of the ship's mooring ropes to the buoys when the sea was not calm. Also the flexible hoses were inclined to give trouble as a result of rough weather. Those were factors affecting the average rate of loading and contributing to the advantage of pier berths to which he had drawn attention. On the other hand, buoy berths had the advantage over loading jetties in the matter of speed of construction since several berths might be established in as short a time as 6 months, whereas the construction of a jetty to serve the same purpose would inevitably take very much longer.

Mr Palmer had asked whether the Authors considered that the fenders had successfully fulfilled their purpose during the time that the piers had been in operation (more than 2 years). The Authors thought it could be safely said that that was the case. As stated in the Paper various small changes had been proposed but they had been definitely of a minor nature.

Regarding the various points referred to by Mr P. R. Robinson, the Authors thought that the " seize up " of some piles at Mina al-Ahmadi was appreciable. It might partly be a chemical action which did not occur everywhere in the sea bed. In the case of some piles it had not been found to have developed completely within a month as suggested. The nature of and rate of increase of the grip of the surrounding strata on piles was a matter regarding which much more evidence was wanted and engineers held different opinions on it.

The Authors agreed with him that the maximum rate of driving piles obtained on the work had been remarkable. In *Fig. 43* (p. 283), the pile-driving barge was shown with the pile-supply barge lashed alongside it and the two moved as one unit. The speed with which the piles had been pitched was attributable to their being steel (so enabling them to be handled with safety and comparative ease) and also to the outrigger under the piles on the supply barge. *Fig. 42* showed a 100-foot-long pile for one of the fender groups at the cargo pier. It had hit the water and was being hoisted by one sling—at its head—for pitching. That sort of thing could not be done with reinforced-concrete piles, because first, they were too heavy, and secondly they would not stand it. Nor could it be done with timber piles with the ordinary type of splicing; and even if long lengths of unspliced wood were used the buoyancy of the piles prevented their dropping so quickly into position in the leaders, which were—as already mentioned in the Paper—of the inside type. Mr Robinson was correct in assuming that the high rate of pile driving had been obtained in calm weather. The easy manoeuvrability of the pile barge and its supply barge could be appreciated by a reference to *Fig. 43*, in which the two rectangles shown at the stern of the pile barge indicated two three-drum diesel-driven winches. The starboard winch worked the two steel quarter-lines with their heavy anchors, which were out on the starboard side, whilst the port winch about 15 feet away did the same on that side. The drive of the winches was very flexible and the two operators had no difficulty in making the barge go sideways, backwards, or diagonally, exactly as the foreman wanted it. The Authors considered the work of the American pile gang the best they had seen in that line of work and would not hope for a better team on any job. They also thought that in considering the speed at which those steel H-piles could be driven, it was necessary to appreciate the speed at which the hammer worked—sixty blows per minute as mentioned in the Paper.

Mr Robinson would be interested to know that no tolerances had been laid down for the spacing of the piles, when they were pitched and driven, but the majority of them were within 3 inches in either direction. There were a few " wild " piles, as one would expect in a maritime structure involving nearly 4,000 piles, and on the approach pier there was one bay which had to be shortened to allow for bad pitching. At that time, however, the weather conditions had been bad and the contractor had not been using the system of crossed quarter-lines for mooring the pile barge. Head and stern lines with a side line at each bow had been used. The piles were, in general, pitched quite accurately, and the Paper described the method used. The fact that the structural members were welded together at site enabled any small inaccuracies to be dealt with easily and rapidly by adjusting the lengths of those members. The unbraced piles were very flexible and fixing them in the desired position did not cause any serious increase in the stresses in them.

The question of barnacle attack in the lower half of the tidal range had been mentioned by Mr Robinson. As he had said, the barnacles (*Balanus amphitrite*, variety *denticulata*) in that zone penetrated the coating and that meant that the enamel on the affected parts only served the purpose of limiting the cathodic current required for a time. That was true, but it limited it at the important time when polarization was taking place, so that probably, although that particular portion of the coating had a life of only about a year, it saved its value in magnesium. The period of 3 years for renewal (mentioned in the Paper) referred to the coal-tar-base paint applied in situ above the pile caps and at the welded bracing connexions below that level and not to the hot-applied coal-tar enamel. It was agreed that the action of barnacles on an enamel coating was largely mechanical.

Mr Robinson had discussed the fire hazard of the timber decking on the oil pier. It was accepted that spilled oil might soak into the timber decking and its joints and form a fire hazard and that that risk was not present with a concrete deck. However, at the pier in question oil spillages had proved to be infrequent and spilt oil was immediately dealt with by the fire personnel present, so that the risk was not a great one. There were arrangements (not shown in Figs 29, Plate 4) for draining the oil hoses and pumping the drained oil into the return salvage line. That salvage line, and also the gratings at deck level and in the large valve platform below, were shown in Figs 29. In connexion with the general question of fire hazard it might be mentioned that Douglas fir was high up in the scale of fire-resistance of various timbers and was in fact classified as having high fire-resistance. Wolman salts had been used for the treatment of decking timber—not with the object of making it fire-resistant but in order to preserve it. It was, however, the case that that treatment added to the fire-resisting qualities. If a piece of wood so treated were put into a fire it would be noticed that the wood did not flame but that smouldering slowly crept along the grain.

The Authors thought that the laminated timber deck was more or less an innovation from the British point of view. On the work under consideration if the 6-inch-by-3-inch or other sections used had been sawn and seasoned to small limits it would, as mentioned, have been possible to get tighter nailing. On the approach pier the resistance to the tractive effort of the rubber-tired vehicles when they went at any speed and, at times, the forces of acceleration or retardation caused the battens gradually to work loose and the noise known as "chatter" resulted. They thought that for the approach pier a concrete deck would have been better but on the oil pier where there was practically no traffic (because for safety reasons vehicles were not allowed upon it when crude oil and similar products were being loaded) it did not matter and the 4-inch deck there was wearing well. The type of deck was, in the Authors' opinion, suitable for such a light flexible structure.

When considering the question of a concrete deck it should also be remembered that when the work had been planned it was not the only development at Kuwait for the Oil Company. The latter was developing the whole oilfield with its many works, building a new town, power station, and many other installations. It had been necessary to allocate the available resources. Concrete had had to be used extensively and there had been difficulty in getting all the aggregate needed. Most of it came from the surface of the desert and had had to be brought from far afield. Moreover (strange though it might seem) well graded sand for good concrete was very difficult to get in the desert. The Company had therefore, spread out its tentacles and made the best of the difficult supply position by taking those suitable materials which were available to it. Although the timber decking had been shipped from the United States it had cost less, when finished, than a reinforced-concrete deck. It consequently had the advantages of being lighter, quicker to construct, more readily available, and cheaper than a concrete deck; and the Authors considered that taking these considerations into account, it was the correct course to use it.

Professor A. L. L. Baker's remarks on fenders were considered very interesting but the Authors thought, however, that the rotating-bell type of dolphin as used at Heysham was not suitable for a light flexible structure with continuous berths on the face. That particular type was both heavy and expensive. As Professor Baker had said, and as was stated in the Paper, the right solution to the problem seemed to be additional longitudinal strengthening for dealing with oblique collisions at the fender groups. The closer spacing of fender groups would, to some extent, prevent oblique collisions occurring on the unprotected face of a berth but there was a practical and economic limit to the number of groups which could be provided. *Figs 56* illustrated the difficulty of guarding against such abnormal berthing. A point of interest regarding the cases which occurred at Mina al-Ahmadi was that on at least one occasion the damage had been caused by an anchor fouling the corner of the fender. The point about the intensive use of the jetty was a valid one and the reply to the Professor's first question was that cases of appreciable damage to the fenders had amounted to only one per 1,530 berthings, which it was agreed was a low incidence. Most modern tankers were all-welded, having very few protusions on the hull, and it was thought that that had to a great extent limited the damage which had been done to the actual rubbing timbers on the gravity-fender faces.

The Authors, as engineers, and not marine men, felt that no jetty yet built had been able to stand up successfully to a large ship which came alongside by charging it, and whatever was done about fenders could not stop that sort of happening. The berthing at the piers described in the Paper was carried out very neatly if one had in mind the huge size of the modern tanker and also the fact that all those ships came into that port in ballast and, therefore, were affected greatly by any wind. The tankers did not usually drop an anchor and they generally came alongside at a small angle of inclination or parallel to the face of the jetty. Two powerful tugs, one at the bow and the other at the stern, were in attendance and finally pushed the side of the tanker with their bows in order to help her alongside. The operation was controlled from the bridge by a berthing master belonging to the port authority—the Kuwait Oil Company. That was the normal method of berthing, but in the early days of operating the oil pier one of the Authors had seen a tanker come straight at the jetty at an angle of 90 degrees. Fortunately the berthing master had been able to have the two anchors dropped in time and the ship had stopped just as she rubbed the top guard rail. But it was not only the big ship which might damage a jetty. At about the same time a little naval craft—about one-tenth the size of a tanker but with a crew five times as many—had come alongside not very gracefully and damage had been done.

With regard to the improvement of the suspension of gravity fenders so that they could move in the longitudinal direction and absorb energy from ships rubbing alongside, and also rotate to some extent about the vertical axis, the proposals shown in *Figs 57* raised some doubt in the minds of the Authors as to their practicability. In lining up the fender-supporting structure at Kuwait they had got within quite accurate limits for maritime work. For example, before hanging the empty cylinders, the top and bottom trunnion hangers had usually been fixed within 1/16 inch of the longitudinal plane and, when tested after the fenders had been hung and the links given their final adjustment by the arrangement indicated in *Figs 52*, any pair of links had hung symmetrically and within 3/16 inch of a vertical plane in the direction of movement. The hanger system provided in the detail design was good. Those adjustments were necessary for the heavy loads to be distributed evenly and for the fenders to work smoothly. If, in addition, further adjustments had to be made so that fenders could rotate as well, it was a matter of asking for something which was considered beyond the limits of accuracy of maritime engineering. It was also thought that such an arrangement would add to the maintenance problem. The Authors would, however, like to learn more about the heavy caisson-type of fender with the rounded face and three-point support shown in *Figs 58*.

They did not consider it possible to make an accurate assessment of the reduction in time of turn-round made possible by the use of the gravity fenders and the consequent saving in shipping costs. That type of fender did allow ships to come alongside in conditions which would not be acceptable for berthing elsewhere. An additional point about them, which had not been mentioned, was that they allowed tankers to lie alongside in sea conditions which would have prevented their doing so had the fenders not been there. The weather was an important factor in the matter and an oil pier with the same facilities, but without gravity fenders, did not exist at the location under consideration and so make a direct comparison possible. It was agreed, however, that such a reduction did to some extent take place.

A number of observations had been taken on the deflexion of the gravity fenders as ships came alongside. The Authors wished to make it clear that those observations, of which copies of two typical records were given in Tables 1 and 2 (pp. 320, 321), had considerable limitations and, therefore, a considerable margin of error. The method had been for an inspector to be on the berth as the ship under observation came in. As it came near he decided which group of fenders would be struck first and quickly got to that point. When the side of the ship was about 10 feet away from the fenders he estimated the distance and started his stop watch. At the moment the first fender was pushed he stopped the watch and noted the deflexion of the cylinder or cylinders from scales which had been painted on the tops of the upper guide-beams. If the ship struck two groups of fenders at the time of making contact the observer was not always able to observe the deflexions at the second group because it was some distance away. Most of the ships observed had a velocity—normal to the face of the berth—of less than 0·75 foot per second as they came alongside, even when the approach appeared to be fast.

Professor Baker's last question on the up-and-down movement of tankers when they were alongside was difficult to answer, because much depended on the size and condition of loading of the ship as well as on the wind and swell conditions. No records were available which would enable a comparison of the type which the questioner had in mind to be made, and it was not possible to say much beyond what had already been stated in the reply to the second question. Tankers appeared, however, to be quite comfortable when there was a 3-foot swell running and the Authors had seen them loading when the swell was 4 feet. In such matters engineers should remember that much depended on the master of a vessel whose judgement of safe conditions for his craft was a very important factor.

The Authors felt that, although Mr Little was impressed by the size of the work, he had not appreciated some of its main points. They could not agree with him that it would have been equally effective if made only 50 feet wide (presumably the oil pier and not the cargo pier was referred to). Such a reduction in the width of the oil pier would have necessitated strengthening it in other ways in order to obtain the same stability and would have reduced the space (45 feet) available for pipelines. Tankers moored on berths opposite one another would have been too close, first, to have suitable breasting moorings when the weather was not good and, secondly, from considerations of safety. The reduction in width of the approach to 15 feet would have made it unsafe in deep water for the heavy vehicles which used it and prevented the two-way traffic required for operational reasons.

Mr Little had recommended a single-slab concrete deck. Apart from the availability of suitable aggregates, already mentioned, he had overlooked one of the first considerations regarding choice of method of construction at such a location. Where a waterless desert site, with scarcely any local resources of power, workshop facilities, or skilled labour, was concerned, it was most essential to choose a type of construction in which as much "manufacture" as possible was done overseas and as little as possible at the site. If that were not done a state of affairs could easily be developed in which so much effort had to be devoted to housing imported skilled workmen and providing food, water, power, and amenities for them, that the effort remaining available for the construction of the project itself would be seriously diminished. In a timber-and-steel structure, even if site-welded, "manufacture" at the site might be said to be limited mainly to welding, an activity involving comparatively few local workmen which could nevertheless be employed on such operations as enamel coating and deck-laying. With a large concrete structure, however, a whole chain of local operations were involved, including obtaining aggregate, crushing, washing, and screening it (the latter without fresh water), the fabrication of shuttering, and steel bending and fixing on a large scale. Some of those processes involved highly skilled imported labour in appreciable numbers. It could therefore be affirmed that the decision to use steel for the structure and timber for the deck was further justified by those considerations. Moreover the adoption of a single-slab concrete deck would have prolonged the construction period and interfered with the essentially flexible nature of the structure which Mr Little appeared to have overlooked. When there was a swell, such as frequently occurred during winter, whilst the south-east wind blew, the piers moved and the movement could be felt when walking on the deck. Nevertheless the work was a very satisfactory structure, considering it as a special installation for the rapid loading of crude oil—and, after all, that was its purpose.

The Authors were also unable to agree with Mr Little's ideas on the size and spacing of gravity fenders. A much greater spacing would increase the possibility of collision with the structure between the fenders. The suggestion that the number of fenders per group should be reduced to two was not understood. Were that done the fenders would have to be increased in size to provide the same degree of resistance. Would that not make the supporting structure more expensive than the design adopted? The Authors were interested to learn that the Admiralty had

TABLE 1.—OBSERVATIONS MADE ON THE BERTHING OF A TANKER

Date: 6.1.50

Section 1 to 4	Tide	Wind	Sea	Ship's Name
	+ 1 Young flood	Southerly Force B	Swell up to 2 ft.	"FERNCOURT" Tonnage: 14,100
5 to 8	Port of Registration	Nationality Owners	Draught	Ballast
	Oslo	Norwegian Diesel Engine	FOR. 9½ ft. AFT 18 ft. Reg. Length: 492·9 „ Breadth: 66·1	2,500 tons (approx.)
9 to 12	Approach Direction to	Obliquity	Velocity	Berth No.
	South	Parallel	25 ft. in 38 sec.	No. 2 (East side of Jetty)

13 *Fender Group Deflexion* :—No. 7 set hit first. All moved 20 inches in this set of three. No. 8 set was hit immediately after.

14 *Remarks* :—Good approach.

TABLE 2.—OBSERVATIONS MADE ON THE BERTHING OF TANKERS

Date : 9.1.50

Section 1 to 4	Tide	Wind	Sea	Ship's Name
	+ 2 Flood tide	Moderate to fresh south-easterly. Force 4–5. Considerable increase since early morning	Waves pronounced with foaming crests and lots of "white horses." Two tugs moored facing south	"FLORCAL" Tonnage : 15,300
5 to 8	Port of Registration	Nationality Owners	Draught	Ballast
	Rouen, France (Not in Lloyds' Register)	French	FOR. XIII AFT XX Dimensions not available	6,000 tons (approx.)
9 to 12	Approach Direction to	Obliquity	Velocity	Berth No.
	South	10° angle from 100 ft. out to within 30 ft. of berth, then parallel	6 ft. in 9 secs. Taken just before striking fender	No. 6 (West side of Oil Jetty)

13 *Fender Group Deflexion* :—Fender Group No. 29. South cylinder 12″ movement eastwards. Centre and north cylinders nil. Fender Group No. 28. South cylinder 3″ east. Centre cylinder 3″ east. North cylinder 3″ east. Fender No. 27 struck immediately afterwards.

14 *Remarks* :—Tanker came in at 10° angle assisted by tug amidships on starboard side. When 20′ from berth rope placed around bollard forward also spring. Fender No. 29 struck by port bow, ropes then placed around bollards aft and stern of Tanker eased over. Fender 28 struck amidships then Fender 27 immediately after by stern. Ship at about 5° angle on first impact. Ship then made fast at berth.

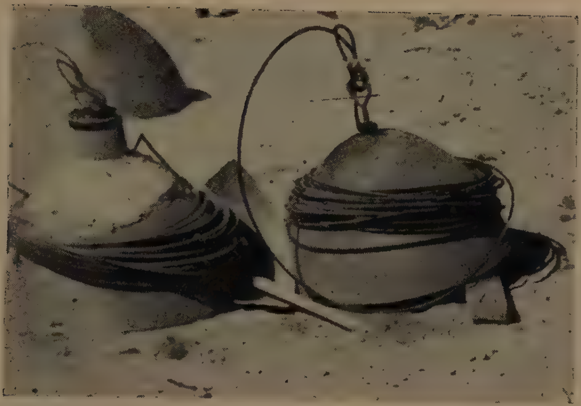
found that the horizontal type of concrete fenders on jetties without low-water bracing had not had satisfactory longitudinal resilience. The concrete-filled fenders at Mina had been called " clumps," which they did not consider to be a suitable name, because well designed fenders were anything but inert masses of material; they were active parts of the jetty and were in fact very much alive. The Authors did not, for one moment, believe that the meeting was the funeral service of gravity fenders! Several types were being designed nowadays and they would be useful in many circumstances, especially in connexion with oil loading and unloading where the distance from the jetty face to the ship was not so important as it was for handling ordinary cargo.

Mr Little's estimate of a maintenance cost of £50,000 per year was considered to be on the high side, the actual figure probably being about half that amount. Amongst other things he had said that one got places on a steel structure where water collected. It might be said of the welded structure under discussion that such water pockets were very much less than in earlier types of steel construction. Incidentally, under-deck bracing was useful for supporting scaffolding for maintenance and *Fig. 62* showed how simple the welded bracing connexions were. Throughout the Paper, the Authors had referred to the jetties as " piers." That might have led Mr Little to a comparison with the seaside pier which he knew. The term " pier " had been used in deference to the American contractors and their staff, who used the word "pier " to mean what in England is called a " jetty ", and who used the term " jetty " for a stone breakwater.

Mr Tomlinson had mentioned certain tests which had been carried out in England in order to ascertain the extent to which different coatings were stripped off piles during driving. From them it appeared that the American enamel was slightly inferior to the brand made by the associated company in Britain. There were of course a number of considerations affecting the decision to use American enamel. The specification was comprehensive and included a variety of tests, and, at the time, the manufacture of the enamel in England had not yet reached the " production " stage. It was, nevertheless, noteworthy that, as stated in the Paper, in no case was the enamel found to be damaged at sea-bed level. Mr Tomlinson had enquired what measures were being taken to dredge the Boat Harbour and to remove the conglomerate which covered a good part of it. Owing to difficulties regarding suitable dredging craft, it was now being removed by grabbing, using explosives where necessary.

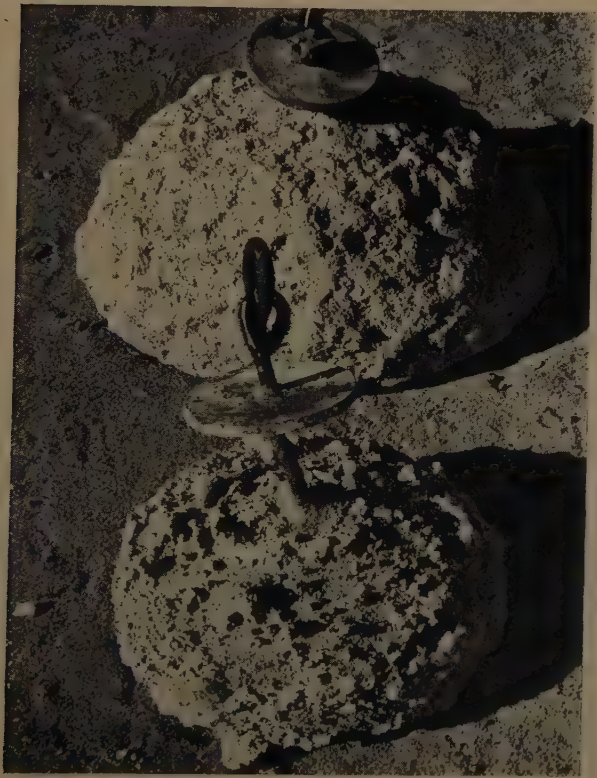
In response to Mr Spencer's question, the Authors were able to confirm that the initial current output was 5½ to 6½ amperes per 200-lb. anode. Mr Spencer had referred to the use of electric-arc welding sets for the supply of current to supplement that given by the anodes in the earlier stages of the cathodic protection. Welding sets had in fact been used experimentally on part of the structure. With regard to the economic

Fig. 59



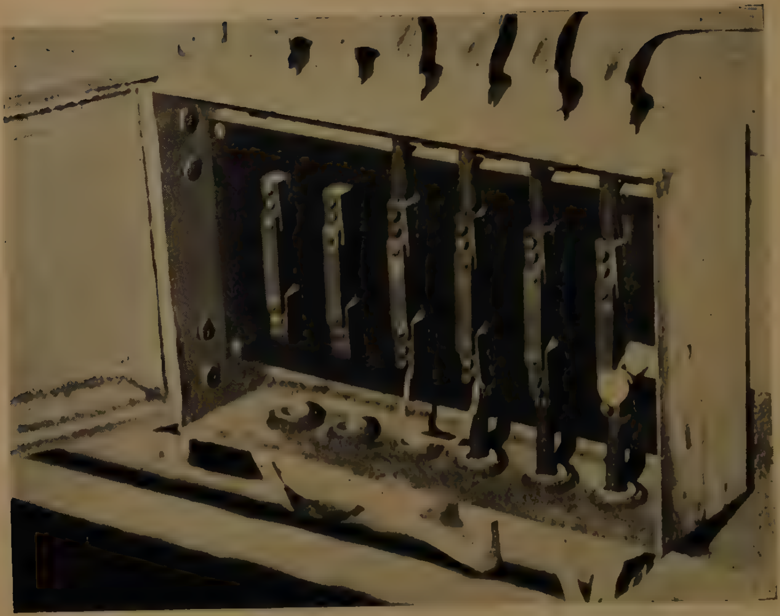
200-lb. Magnesium Anodes before Immersion. Note the Bitumen Collar on the Left-Hand Anode

Fig. 60



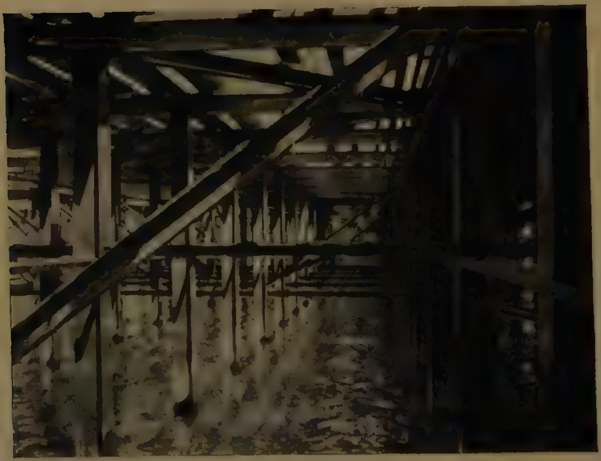
Magnesium Anodes taken from the Sea Bed after Immersion for Some Time. Original Weight: 200 lb.

Fig. 61



Six-Way Junction Box Fixed at Deck Level to Control Group of Magnesium Anodes

Fig. 62



Typical View showing "Clean" Appearance of Under-Deck Bracing aspects of cathodic protection, the approximate cost of hot coal-tar-enamel coating below the half-tide level was £34 per pile. The Authors would, however, remind Mr Spencer that that coating had not been stripped during driving although some had been removed from between low-water and half-tide levels by barnacles and that comparison of costs of cathodic protection under different conditions of the protected structure was difficult. However, on the basis of a period of 10 years, and assuming progressive deterioration of the coating during that period, it was estimated approximately that the cost of magnesium anodes during the period would be of the order of £19 per coated pile whilst the corresponding cost per completely uncoated pile was £82. Thus the cost of protection with no coating would substantially exceed the cost of protection with coating over that period. Those costs were high but, as Mr Spencer had remarked, economy would be effected by the use of electrical power, which had been planned. *Figs 59, 60*, and *61* showed some further details of the installation referred to in Figs 41, Plate 4.

Mr Jellett had questioned the accuracy with which a tanker needed to tie up. The latitude provided by the hoses and the slewing of the hose derricks was considerable. However, it was advantageous to come as near to the correct position as possible and observation of the tankers had shown that they seldom had any difficulty in getting within a foot or two of where they needed to go. The Authors considered that Mr Jellett was very optimistic if he thought, first, that he could get underwriters to lay down all the conditions which could exist when catastrophes occurred between ships and their berths and, secondly, that engineers could design fenders which met the needs of the infinite number of possible incidents which might happen but would not meet the catastrophic conditions. Their view was that engineers must continue to improve fendering as much as was practicable. Mr Waddington had, however, expressed a point of view opposite to that of the super-efficiency suggested by Mr Jellett. The Authors agreed that the appearance of a solid masonry structure might have some psychological effect on those in charge of ships coming alongside and make them more cautious than if the berth were an open structure. For open structures they preferred, however, to rely on good fenders and would remind Mr Waddington that when a ship came alongside a solid quay wall there was a "squeezing out" of the water between the two which provided some fendering action. The camouflage suggested for the Mina al-Almadi piers would not work, because many of the tankers were frequent visitors to the port and might even show their resentment at being hoodwinked!

Mr J. A. Williams had referred to the term "dead-weight" used on p. 263. A correction had been made since the discussion took place, by means of a footnote on that page. As he had said, such calculations were extremely arbitrary, because so many factors were involved. At the piers under discussion, oil loading was the main function and fully loaded tankers did not usually come alongside. The arbitrary displacement of 30,000 tons, would, therefore, cover the cases of much larger tankers berthing in ballast conditions and, judging by the operational results, was satisfactory. Mr Williams would be interested in the remarks made regarding the berthing observations in the reply to Professor Baker and in the observations given in Tables 1 and 2.

Regarding the pulling tests, Mr Brown had asked if any details of the incremental rises of the piles could be given. As stated in the Paper that particular group of piles had been driven at the low-water line by the floating rig working at high tide so that the driving conditions would be similar to those prevailing when the permanent piles were driven. The work of pulling those piles had to be done at the lowest tides available so that there would be enough time to complete a pull before the returning tide stopped the work. That necessitated commencing each test in the dark and prevented any levelling being done. As Mr Brown had pointed out, the two lines in *Fig. 37* (*b*) should be connected with a loop similar to a hysteresis curve and that Figure had now been revised. With reference to the deflexion test, the length of the pile (No. I—10—F)—from ground level to top at the time of testing—was less than one-half that of the piles in the piers. A designed deflexion of 6 inches was provided for on the long piles, but an increased deflexion was given to the test pile in order to verify the position below the sea-bed of the point of fixity; that was the object of the test. There might have been a " permanent " displacement of about an inch at the top of the test pile but the Authors, after taking into consideration the flexibility of such piles and the conditions under which the test had been carried out, did not attach any special significance to that. It was regretted that circumstances had not permitted the repetition of the loading of that test pile.

In conclusion, the Authors wished to pay a tribute to the American contractors who had been engaged on the work. They had been good and the Authors had formed a very high opinion of the American workmen employed.

Correspondence on this Paper is now closed and no further contributions can be accepted.—SEC. I.C.E.

ROAD ENGINEERING DIVISION MEETING

24 January, 1952

Brigadier A. C. Hughes, C.B.E., T.D., B.Sc., M.I.C.E.. Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Road Paper No. 36

"Some Theoretical Aspects of Road Traffic Research"*

by

John Glen Wardrop, B.A.

SYNOPSIS

Some of the mathematical and statistical aspects of the disposition and behaviour of road traffic which are of importance in research are considered. It is shown that vehicles can simultaneously be regarded as distributed at random along a road and in time. Frequency distributions of speed for a given traffic stream are of two kinds, one associated with successive vehicles passing a point and the other with successive vehicles along a road at an instant. The corresponding average speeds generally differ by 6 to 12 per cent.

A formula is given for the frequency with which vehicles would overtake one another if there were no interference with overtaking.

In considering the capacity of road systems it should be remembered that increases in the amount of traffic (the flow) generally produce corresponding decreases in speed. Capacity can be defined as "the flow which produces the minimum acceptable average journey speed."

Delay and capacity are discussed in relation to traffic signals, and it is shown that the shortest practicable cycle does not necessarily result in the minimum average delay. The randomness of traffic can cause very long delays at traffic signals.

When there are alternative routes which a traffic stream can follow, it may divide itself between them. The consequences of two possible rules governing this division are considered.

"Before-and-after" studies are discussed, and a statistical test for the genuineness of a difference in, say, average journey time "before" and "after" is given. A method of determining the number of observations required is also given.

INTRODUCTION

Road engineers and research workers are tackling a large variety of problems directly concerned with the movement of vehicles. Their methods include "before-and-after" studies of speed for changes in road lay-out, control or traffic composition, general surveys of traffic conditions, and investigations into the effect of such factors as parking, road width,

---

* Crown Copyright reserved.

right turns, and lane markings on the behaviour of traffic. A number of theoretical problems occur in this work, and the object of the present Paper is to consider some of these problems. The Paper is concerned with some of the mathematical and statistical aspects of the disposition and behaviour of traffic on open roads and at intersections, including the questions of capacity, the use of alternative routes, and the planning and interpretation of "before-and-after" studies. The treatment is mainly statistical because the variability of vehicle behaviour often demands this approach, but statistical ideas have been introduced from first principles so far as possible. An attempt has been made to illustrate the application of some of the theory, although this is not always possible.

A glossary of terms and a list of the symbols used most frequently in the Paper are given in Appendix I. In general, when formulae are quoted, no units are mentioned, since the relations hold between the physical quantities concerned. When interpreting a formula it is, of course, necessary to use the same units of length and time throughout.

The Value of a Theoretical Approach

It is not always appreciated that in a severely practical subject such as traffic engineering there is need for theory. However, the history of science suggests that progress in any field of research can best be achieved by a judicious mixture of practical experience, experiment, and theory. Although a particular problem, such as that of assessing the saving in vehicle-hours due to widening a given road, can be solved by direct observation, a theoretical background is required if the observations are to be economical and useful and their interpretation valid. Moreover, the effect of road widening can be studied at a number of sites each with a different set of conditions, but it is a theoretical problem to generalize the results to cover conditions not actually observed. Theory may also be useful in suggesting the type of relation to be expected and the order of the result.

It must be emphasized that theory should not be divorced from experiment. A purely theoretical approach to a traffic problem usually has to be very much simplified or else it involves very difficult mathematical analysis. But simplifying assumptions can be tested by experiment, and empirical relations can be found by applying theoretical methods to data.

FLOW AND SPEED OF TRAFFIC

A necessary condition for many theoretical investigations into road traffic problems is that the way in which vehicles are distributed along the road should be specified at all times considered. That is to say, a theoretical model of traffic is required. Now Adams [1] has shown that the times at which vehicles pass a given point, in the absence of a nearby intersection or other interruption of the traffic stream and in conditions of moderate flow,* may be regarded as a random series. He also remarked that traffic could be regarded as a random distribution in space along the road. Following Adams, a series of events in time is defined as "random" when:

---

[1] The references are given on p. 354.

(*a*) each event is completely independent of any other event;
(*b*) equal intervals of time are equally likely to contain a given number of events.

A random distribution in space is defined by the corresponding conditions in terms of space instead of time.

It may not perhaps be clear that a theoretical model of traffic can be set up in which it is randomly distributed in both time and space, but this is the case. Consider a random series formed by the times $\ldots t_{-2}, t_{-1}, t_0, t_1, t_2, \ldots$ at which vehicles pass a point O on the road. Suppose, first, that all the vehicles have the same speed v. Then at any instant t, the vehicles will be at distances $\ldots v(t-t_{-2}), v(t-t_{-1}), v(t-t_0), v(t-t_1), \ldots$ along the road, measured from O. Since the space-intervals between these points are merely the original time-intervals multiplied by v, they form a random distribution in space.

Consider now a traffic stream composed of a number of subsidiary streams, in each of which all the vehicles are travelling at the same speed and form a random series. Since the combination of two random series or distributions in the above sense is itself a random series or distribution, the whole stream is random in both space and time. Moreover, the speeds of successive vehicles in space or in time form a sequence of random variables, each being independent of its predecessor.

Distribution of Speed in Time

Suppose that there are subsidiary streams, with flows $q_1, q_2, \ldots q_C$ and speeds $v_1, v_2, \ldots v_C$. Let the total flow be given by

$$Q = q_1 + q_2 + \ldots + q_C$$
$$= \sum_{i=1}^{C} q_i$$

and let $f_1 = q_1/Q$; $f_2 = q_2/Q \ldots$; and $f_C = q_C/Q$

Then the numbers $f_1, f_2, \ldots f_C$ are the frequencies in time of vehicles whose speeds are $v_1, v_2, \ldots v_i$ and $\sum_{i=1}^{C} f_i = 1$

Distribution of Speed in Space

Consider the subsidiary stream with flow q_i and speed v_i. The average time-interval between its vehicles is evidently $1/q_i$, and the distance travelled in this time is v_i/q_i. It follows that the density of this stream in space, that is to say, the number of vehicles per unit length of road at any instant (the concentration), is given by

---

* The term "flow" is used in this Paper in the sense of quantity of traffic (vehicles per hour) in preference to "volume."

$$k_i = q_i/v_i, \; i = 1, 2, \ldots C \quad \ldots\ldots \quad (1)$$

The quantities $k_1, k_2, \ldots k_C$ represent the concentrations of vehicles in each individual stream and the total concentration is given by

$$K = \sum_{i=1}^{C} k_i$$

Putting $f_i' = k_i/K$ gives the frequencies $f_1', f_2', \ldots f_C'$ of $v_1, v_2, \ldots v_C$ in space.

The following table illustrates these ideas for some actual data. In practice, speeds vary continuously, but they have been grouped in 4-mile-per-hour groups.

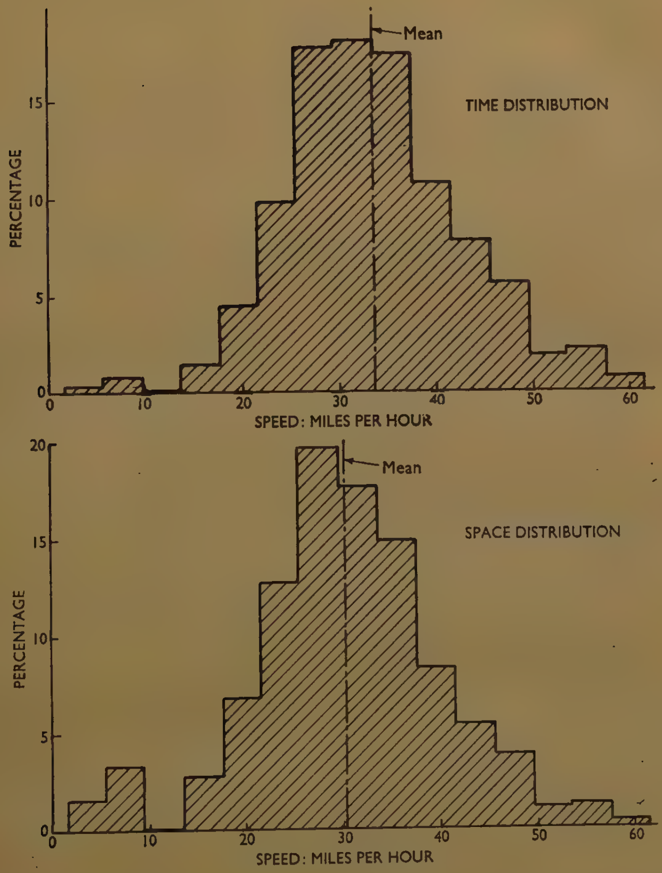
TABLE 1.—DISTRIBUTIONS OF SPEED IN SPACE AND TIME. WESTERN AVENUE, GREENFORD (MIDDLESEX). DUAL CARRIAGEWAY. EAST-BOUND TRAFFIC ONLY

Speed range m.p.h.	Flow v.p.h.	Percentage in time	Concentration v.p. mile	Percentage in space
*				
2– 5	1	0·3	0·3	1·7
6– 9	4	0·8	0·5	3·3
10–13	0	0·0	0·0	0·0
14–17	7	1·5	0·4	2·9
18–21	20	4·5	1·0	6·9
22–25	44	9·9	1·9	12·7
26–29	80	17·9	2·9	19·7
30–33	82	18·2	2·6	17·7
34–37	79	17·5	2·2	14·9
38–41	49	10·9	1·2	8·3
42–45	36	7·9	0·8	5·4
46–49	26	5·7	0·6	3·8
50–53	9	1·9	0·2	1·1
54–57	10	2·2	0·2	1·2
58–61	3	0·8	0·1	0·4
Total	450	100·0	14·0	100·0

* Strictly 1½–5½, 5½–9½, etc.

The two distributions are illustrated in *Figs 1.* It will be seen that there is a distinct difference between them; the bulk of the time-distribution is further to the right than that of the space-distribution, meaning that speeds are greater on the whole for the time-distribution. There will always be such a difference if there is any variation in speed.

Figs 1



Time and Space Distributions of Speed. Western Avenue, Greenford (Dual Carriageway)

Mean Speeds

With each of two distributions of speed there is associated a mean value. They are given by the following formulae :

$$\text{Time-mean speed : } \bar{v}_t = \sum_{i=1}^{C} q_i v_i / Q = \sum_{i=1}^{C} f_i v_i \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (2)$$

$$\text{Space-mean speed : } \bar{v}_s = \sum_{i=1}^{C} k_i v_i / K = \sum_{i=1}^{C} f_i' v_i' \quad . \quad . \quad . \quad . \quad . \quad . \qquad (3)$$

But from equation (1)

$$k_i v_i = q_i \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

and therefore

$$\bar{v}_s = \sum_{i=1}^{C} q_i/K = Q/K$$

Hence

$$Q = K\bar{v}_s \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

It may be noted that there is no equivalent relation involving the time-mean speed.

It is helpful to consider how these alternative mean speeds might arise in practice. Speeds at a point on the road can be measured by any of the following methods:

(1) Timing successive vehicles, or a representative selection (for example, every 5th, or those whose registration numbers end in 2), over a short measured distance of length l, and calculating $v = l/t$ for each, where t is the time taken. The mean is $\bar{v} = \Sigma v/n$ where n denotes the number of vehicles.

(2) Use of the Radar Speedmeter, which gives a direct reading of speed for each vehicle. The mean is $\bar{v} = \Sigma v/n$.

(3) Timing vehicles over a short distance as for (1), but calculating first the average time taken, $\bar{t}$, and then $\bar{v}' = l/\bar{t}$ as the mean.

(4) Taking two aerial photographs of a uniform road at a short interval τ, measuring the distance covered by each vehicle, x, and computing $v = x/\tau$. The mean is $\bar{v}' = \Sigma v/n$.

Of these methods, (1) and (2) give the time-mean speed, and (3) and (4) give the space-mean speed.

It is, of course, possible to calculate either the time- or the space-mean speed from a set of readings of speed, if it is known how they were obtained and therefore which type of distribution they form. Thus, if $v_1, v_2, \ldots v_n$ are obtained from a Radar Speedmeter they form a time-distribution, and the time-mean is $\Sigma v/n$, whilst the space-mean is given by

$$\bar{v}_s = \frac{n}{\Sigma(1/v)}$$

This is, of course, the harmonic mean of the v's. On the other hand, if they are obtained from aerial photographs or by a similar technique they form a space-distribution, $\Sigma v/n$ is the space-mean, and the time-mean is

$$\bar{v}_t = \frac{\Sigma v^2}{\Sigma v}$$

It is shown in Appendix II that the general relation between time- and space-means is

$$\bar{v}_t = \bar{v}_s + \frac{\sigma_s^2}{\bar{v}_s} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where σ_s is the standard deviation * of the space-distribution, defined by

$$\sigma_s^2 = \sum_{i=1}^{C} k_i(v_i - \bar{v}_s)^2/K \qquad (7)$$

and measured in the same units as the speed. If c_s is the coefficient of variation of the space-distribution of speed, defined as the ratio of the standard deviation to the mean (that is to say, $c_s = \sigma_s/\bar{v}_s$)

then $$\bar{v}_t = \bar{v}_s(1 + c_s^2) \qquad (8)$$

As an illustration of this result, consider the example of Table 1, where the following values are found :

	Mean speed : m.p.h.	Standard deviation : m.p.h.	Coefficient of variation
Time :	33·5	9·1	0·27
Space :	30·1	10·1	0·34

It can be verified that equation (8) holds in this case. The equation shows that in every case the time-mean speed is greater than the space-mean speed, unless there is no variation in speed, when $c_s = 0$. The coefficient of variation is usually between 0·25 and 0·35 so that the time-mean is 6 to 12 per cent greater than the space-mean.

In a particular investigation, it may not be very important which of the two means is used, although it will be shown below that some cases call for one and some for the other. But it is most important that the same mean should be used throughout any investigation, so that all comparisons are fair. There is a danger that a comparison of mean speeds measured some years apart or by different investigators will be invalid because they are not of the same kind.

Speed versus Journey Time

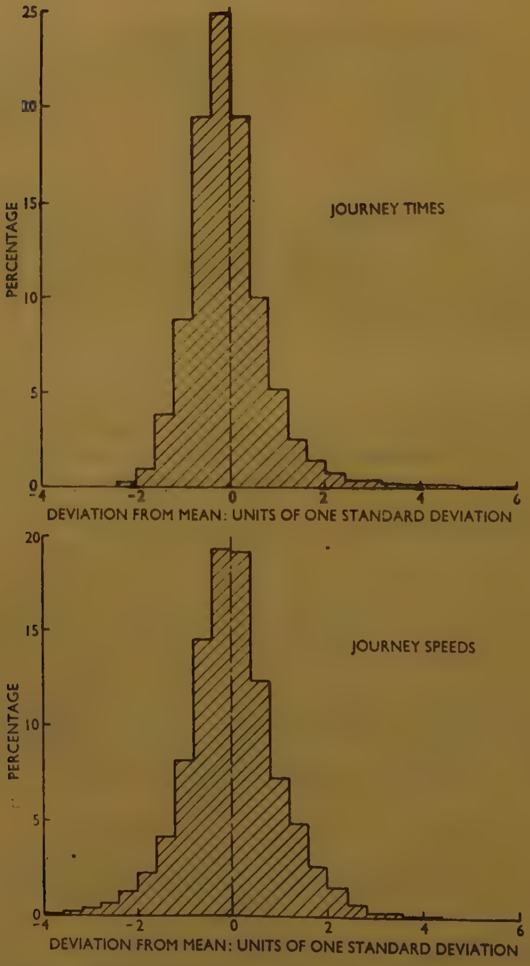
The choice between time- and space-mean speeds is really one between speed and journey time. Fundamentally it is the characteristics of successive journeys on a given route that are required, and these can be analysed in terms of speed, giving the time-mean speed, or in terms of time, giving the mean journey time, which by division into the length of the route gives the space-mean speed.

Now time is a directly measurable physical quantity. On any route or section of road, the length is a fixed quantity, while the duration of the journey varies. Times on successive sections are additive whereas speeds are not. The delay caused by two equal intersections is twice the delay caused by one, provided that they are sufficiently far apart. In planning a journey, one wishes to know how long it will take rather than what the average speed will be. Also the cost of slow movement is measured in terms of time. There is therefore much to be said for using average times in preference to average speeds when assessing the effect of a change.

---

* In general, if a frequency distribution consists of quantities $x_1, x_2, \ldots x_C$ with frequencies $f_1, f_2, \ldots f_C$, and the mean is $\bar{x} = \sum_{i=1}^{C} f_i x_i$, the standard deviation σ is a measure of dispersion, defined by the equation $\sigma^2 = \sum_{i=1}^{C} f_i (x_i - \bar{x})^2$.

Figs 2



Frequency Distributions of Journey Time and Speed in Central London

On the other hand speed measurements are frequently more consistent. It has been found that journey times often have very skew distributions with a long " tail " consisting of very slow journeys, whereas the corresponding distributions of speed tend to be more symmetrical. A typical example is shown in *Figs 2*, where the frequency distributions of journey time and journey speed for roads in Central London are plotted. It was also found from these data that the coefficient of variation was 20 to 25 per cent less in the case of speeds. This is an important result, since it means that only about 60 per cent of the number of runs are needed to give the same proportional accuracy in the mean speed as in the mean journey time.

The results quoted above were obtained on main roads in London, with a high concentration of controlled intersections (about 5 per mile). The more intersections there are per mile the greater is the tendency to skewness in the journey-time distribution. Where intersections are less dense, the journey-time distribution may be reasonably symmetrical, and so little is to be gained by converting to speeds.

FREQUENCY OF OVERTAKING

Assuming that there is no interference with overtaking, the frequency with which vehicles overtake one another can be derived from the speed distribution. Suppose that $v_1 < v_2 < \ldots < v_C$, and consider a vehicle with speed v_i. The speed relative to it of a subsidiary stream whose speed is v_j, where $j > i$, is $v_j - v_i$, and their concentration is k_j. Hence the average distance between them is $1/k_j$, and therefore they pass the vehicle with speed v_i at intervals of $1/(v_j - v_i)k_j$. Hence the frequency of overtaking experienced by this vehicle is

$$\sum_{j=i+1}^{C} k_j(v_j - v_i),$$

the summation extending over all speeds faster than v_i. Now there are k_i of these vehicles per unit length of road, each being overtaken at the average rate given above. Hence the total number of overtakings in which the slower vehicle has speed v_i is $k_i \sum_{j=i+1}^{C} k_j(v_j - v_i)$ per unit length per unit time. Summing over $i = 1, 2, \ldots C$ gives the total number of overtakings as $\sum_{i=1}^{C-1} \sum_{j=i+1}^{C} k_i k_j (v_j - v_i)$ per unit length per unit time. In terms of the frequencies in space (f_i') and time (f_i) respectively, this becomes:

$$\text{number of overtakings} = K^2 \sum_{i=1}^{C-1} \sum_{j=i+1}^{C} f_i' f_j' (v_j - v_i) \text{ per unit length per unit time} \quad . \quad . \quad . \quad (9)$$

$$= Q^2 \sum_{i=1}^{C-1} \sum_{j=i+1}^{C} f_i f_j \left(\frac{1}{v_i} - \frac{1}{v_j} \right) \frac{\text{per unit length}}{\text{per unit time}} \quad . \quad . \quad (10)$$

If there is interference with overtaking, expression (10) can be taken as the number of desired overtakings per unit length per unit time, provided that the distribution of speeds is determined when the flow is low, so that it represents desired speeds. Thus for a given distribution of speeds the number of desired overtakings increases as the square of the flow.

In the particular case where the space-distribution of desired speed is approximately "normal" *, with standard error σ_s, by replacing the discontinuous distribution of speeds by a continuous one the density of overtaking can be expressed as:

$$\text{number of overtakings} = \frac{Q^2\sigma_s}{\bar{v}_s^2\sqrt{\pi}} = 0{\cdot}56\,\frac{Q^2\sigma_s}{\bar{v}_s^2}\,\frac{\text{per unit length}}{\text{per unit time}} \quad . \quad (11)$$

For instance, in the example quoted in Table 1, assuming that the space-distribution is approximately normal, there would be a density of desired overtakings of 1,090 per mile per hour. The actual number, of course, would generally be smaller owing to interference, and the ratio of actual to desired overtakings could be regarded as an index of traffic congestion.

Capacity of Road Systems

The subject of the capacity of road systems has been discussed for many years. Numerous estimates of the capacity of a traffic lane have been based on the assumption that each vehicle follows the one ahead at exactly the same speed and at the minimum safe headway or the minimum headway acceptable to the average driver. However, these conditions are extremely artificial and take little account of what drivers actually do. They may apply to some extent at intersections but on a section of road free from controlled intersections entirely different considerations apply. An alternative approach is needed; references to a suitable method have already been made by Glanville [2,3] and Smeed,[4] but this method will be discussed more fully here.

Owing to the randomness of the traffic and the variability of its speeds, in moderate traffic there is a continual shuffling process and overtaking is frequent. If the flow is greater, there is more interference with overtaking, and the average speed is less. It is more appropriate to regard the speed as a function of the flow than vice versa.

---

* The "normal" frequency distribution is symmetrical about a single peak, and if $\bar{x}$ is the mean and σ the standard deviation the frequency in the range x to $x + dx$ is $\frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{1}{2}(x-\bar{x})^2/\sigma^2} dx$.

This point of view may be unfamiliar to road engineers, and perhaps requires some elaboration. One difficulty is that as congestion increases a time is reached when traffic stops completely, although only temporarily. At such a time the speed is zero and so is the flow, a state of affairs which does not conform with the idea of speed decreasing as flow increases. However, average conditions are more important than chance fluctuations in the long run. It is helpful to consider an actual experiment to determine the relation between speed and flow on a given section of road in one direction. Any delays caused by conditions in this section ought to be included, although some of these delays may occur outside it. The experiment can only be made satisfactorily if there is a sufficiently free approach, which will not become congested before the section being studied does, and which can accommodate all vehicles held up by congestion in the section. Also there must not be a bottleneck beyond the section which can react on it. These are extremely practical problems when heavily intersected roads are being studied and make the isolation of road sections very difficult. However, suppose that it can be done, and that either the inflow to the section can be controlled or that it varies with time in a suitable way. Then by taking a timing point on the approach road well outside the section, so that all vehicles waiting to enter it are included, and another timing point at the end of the section, values of average speed (or journey time) and inflow can be obtained for a series of periods. If the flow increases the average speed will generally fall (sometimes imperceptibly at first—see below). Eventually a stage will be reached at which the slightest increase in flow will cause a queue * to accumulate at some point in the section. This may be called the saturation level of flow. It could be taken as the capacity of the road, but as it corresponds to a position of instability, and may produce very low speeds indeed, this is scarcely appropriate.

In practice, of course, it may be impossible to isolate a road section in the way described. If so, it is not possible to define its capacity since its behaviour is governed by that of neighbouring sections. In such a case the capacity of a longer section should be considered.

Capacity of an Open Street

An example of the results actually obtained is shown in *Fig. 3* where values of running speed (speed while in motion) are plotted against the flow on the whole street for groups of main streets of varying width in Central London. These results have been given previously by Glanville.[3] It will be seen that, in general, as the flow increases the speed is not appreciably influenced at first, but beyond a certain point the speed decreases steadily with increasing flow. In the case of the narrow streets the speed begins to decrease at once. It has been found possible to approximate to these results by the following empirical formula :

---

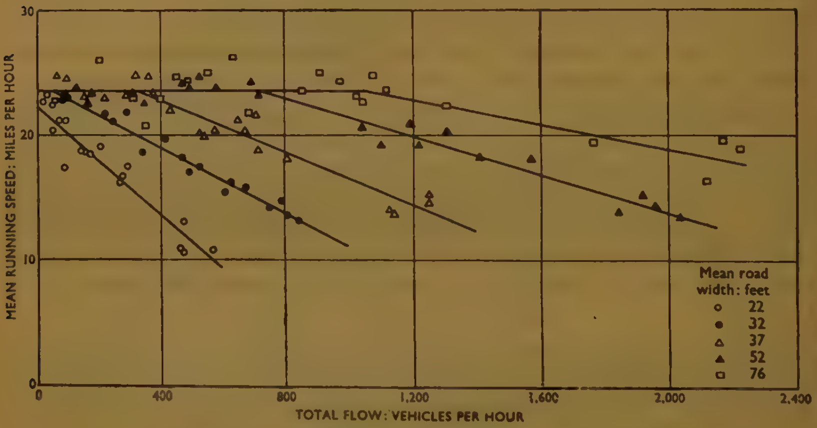
* A queue is here understood to be a chain of vehicles at minimum headway whether stationary or moving.

$$v_r = 31 - \frac{q + 430}{3\,(w - 6)} \text{ or 24 m.p.h., whichever is less,}$$

where v_r denotes space-mean running speed in miles per hour.
q „ total flow in vehicles per hour
w „ road width in feet,

provided that the speed given by the formula is not less than 10 miles per hour. However, it must be regarded as a convenient accident that the speed/flow relation is approximately linear within certain wide ranges. Normann [5] gives a single straight-line relation between speed and flow on four-lane open divided roads in the U.S.A. throughout the range from zero to 4,000 vehicles per hour, but since he gives no points it is not possible to judge how well this relation fits the facts.

Fig. 3



Relation between Mean Running Speeds and Total Flow in Central London

Given the relation between speed and flow, capacity can be defined as "the flow which produces the minimum acceptable journey speed." The average capacity so defined can be calculated for these Central London streets from the above formula. For instance, if the minimum acceptable speed is 15 miles per hour, the average capacity of a 30-foot street is about 700 vehicles per hour. In other words, if 700 vehicles require to use such a street, conditions will allow them to travel at an average speed of 15 miles per hour. The corresponding average capacities for 40- and 50-foot roads are 1,200 and 1,700 vehicles per hour respectively.

Theoretical Approach to Capacity of an Open Road

By the use of the ideas mentioned earlier, and some assumptions about the time required to overtake a vehicle, it should be possible to deduce a theoretical speed/flow relation for an open road. Some work on these lines has been done by Kinzer,[6] but he restricted himself to the question of the distance which a vehicle can expect to travel without interference. This is clearly related to the average speed, but more work is required before the speed itself can be found. It appears that the problem could be solved in a particular case by a somewhat tedious computation, but the derivation of a general expression appears to be difficult.

SIGNAL-CONTROLLED INTERSECTIONS

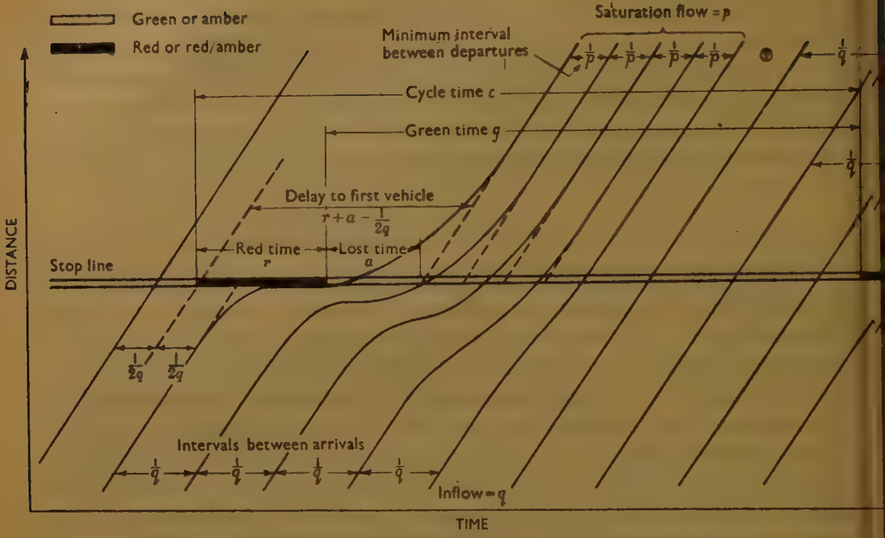
In the case of an intersection, theoretical treatment is easier, and Clayton[7] has given several formulae for cycle time, capacity, and delay at traffic signals. By following his calculations with slightly more general values, the results given below for a 2-phase signal can easily be obtained. It is assumed that the intersection and its traffic are "symmetrical"; that is to say, that the flow from, say, the north equals that from the south and the corresponding approach roads are of equal width, and similarly for the east-west direction. It is also assumed that the flow is uniform, that is to say, the intervals between the successive vehicles are all the same for the period studied. Let the following symbols be defined:

	Phase 1	Phase 2	Total
Flow	q_1	q_2	Q
Saturation flow .	p_1	p_2	
"Lost" time .	a_1	a_2	A
"Green" time .	g_1	g_2	$c =$ cycle time
"Red" time .	$r_1 = g_2$	$r_2 = g_1$	

These symbols can be interpreted by reference to *Fig. 4*. On each phase, vehicles arrive with inflow q, the appropriate subscript being added, and hence at intervals of $1/q$. Some are stopped during the red period, of length r, including the red/amber. At the green signal, the first vehicle leaves, but it is delayed by an additional amount a due to its acceleration. After this vehicle, however, successive vehicles follow at equal intervals of $1/p$ until no more are waiting. This is the period when vehicles are following one another at minimum achievable headway, and the flow p is called the "saturation flow." During the remainder of the "green" period vehicles arrive and leave at intervals of $1/q$, and experience no delay.

With this notation q and p take the place of Clayton's D, the density of traffic (flow per lane), and S, the saturation density. In fact, if there are n lanes, $q = nD$ and $p = nS$. It is more convenient to work in terms of flow rather than density, because traffic does not always divide up into a fixed number of lanes, even at a controlled intersection. The formula given by Clayton[7] for the minimum cycle which can accommodate the traffic without the formation of a steadily increasing queue on either phase, generally known as Adams' Formula, becomes

Fig. 4



Distance/Time Diagram for Signal-controlled Intersection

$$c = \frac{A}{1 - \frac{q_1}{p_1} - \frac{q_2}{p_2}} \quad . \; . \; . \; . \; . \; . \; . \quad (12)$$

A formula for the average delay to traffic on phase 1 is

$$t_1 = \frac{\left(r_1 + a_1 - \frac{1}{2p_1}\right)^2}{2c\left(1 - \frac{q_1}{p_1}\right)} \quad . \; . \; . \; . \; . \; . \quad (13)$$

This is almost identical with that given by Clayton ; for the sake of completeness the derivations of both of the above expressions are given in Appendix III.

In the expression for average delay quoted above, the term $1/2p_1$ is generally small compared with $r_1 + a_1$ and can be neglected. Hence the average delays on each phase on a 2-phase signal can be approximately represented by

$$\left.\begin{aligned} t_1 &= \frac{(r_1 + a_1)^2}{2c\left(1 - \frac{q_1}{p_1}\right)} \\ t_2 &= \frac{(r_2 + a_2)^2}{2c\left(1 - \frac{q_2}{p_2}\right)} \end{aligned}\right\} \quad . \; . \; . \; . \; . \; . \quad (14)$$

The quantities $r_1 + a_1$, $r_2 + a_2$ may be called the "effective red times" for the two phases.

The average delay for the intersection as a whole is given by

$$T = (q_1t_1 + q_2t_2)/Q \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

the average delay on each phase being weighted by the appropriate flow

Hence
$$T = \frac{1}{2Qc}\left\{\frac{q_1(r_1 + a_1)^2}{1 - \dfrac{q_1}{p_1}} + \frac{q_2(r_2 + a_2)^2}{1 - \dfrac{q_2}{p_2}}\right\} \quad . \quad (16)$$

Optimum Phase Times for a Two-Phase Fixed-Time Signal

A reasonable criterion for the optimum settings of a traffic signal is that the average delay to all vehicles should be the minimum. In most practical examples the optimum cycle in this sense is also the minimum cycle. However, it is shown in Appendix IV that this is not necessarily the case. The conditions in which it is so are given, and in the other cases the cycle which gives the minimum average delay is specified. An example of the latter case, in which the flow is assumed to be uniform, is given below:

	Phase 1	Phase 2	Total
Lost time (a), secs.	10	10	20
Flow (q), v.p.h.	1,000	200	1,200
Saturation flow (p), v.p.h. .	5,000	2,000	

Comparison of Minimum and Optimum Cycles

Phase 1	Phase 2	Time: seconds Minimum cycle	Optimum cycle
Green	Red . .	10	12
Amber	R/Amber.	3	3
Red	Green .	13	33
R/Amber	Amber .	3	3
Cycle time		29	51
Mean delay: phase 1 .		11	8
Mean delay: phase 2 .		13	23
Overall mean delay. .		11·6	10·3

It will be seen that the average delay can be reduced by 11 per cent if the cycle is increased by 75 per cent. This sort of situation may arise when one phase refers to a minor road. It is not known whether a similar situation can arise in the case of random traffic.

Vehicle-Actuated Signals

Clayton[7] has given a formula for the average delay to a light flow of vehicles on, say, a minor road at vehicle-actuated signals, if the traffic is random but there is no " maximum green " period on the major road. Garwood[8] has given the average delay to the same vehicles in the more general case where traffic is random and there is also a " maximum green " period on the main road. In the general case the problem of calculating average delay becomes very complicated mathematically and so far no solution has been worked out.

It is possible to give a very approximate solution assuming uniform flow and also an " ideal " vehicle-actuated signal. It is assumed that this signal adjusts itself to the minimum cycle as the flow varies. Then it is shown in Appendix V that in the case of a symmetrical intersection, where $q_1 = q_2 = q$, $p_1 = p_2 = p$ and $a_1 = a_2 = a$, the average delay per phase is given by

$$t_v = \frac{a\left(1 - \frac{q}{p}\right)}{1 - \frac{2q}{p}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

This can be compared with the corresponding formula for a fixed-time signal obtained from (14):

$$t_f = \frac{(r + a)^2}{2c\left(1 - \frac{q}{p}\right)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

These two formulae for delay are plotted against flow for a particular example in *Fig. 5*. It will be seen that the vehicle-actuated signal is very much better than the fixed-time variety until saturation is nearly reached.

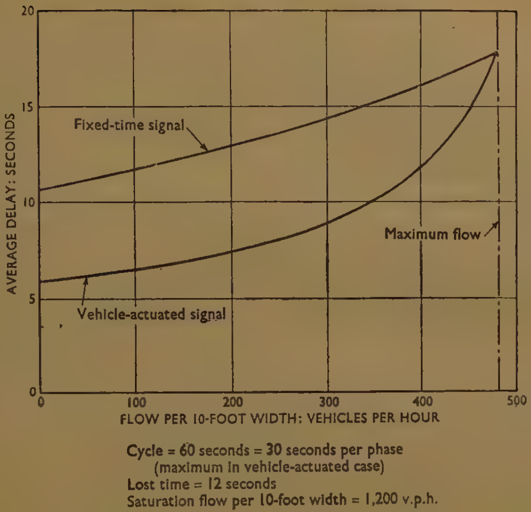
The assumption made above about the behaviour of the vehicle-actuated signal is most applicable to a heavily trafficked signal controlled by a traffic integrator which adjusts the cycle in relation to the number of vehicles entering the intersection in accordance with Adams' Formula, although this type is usually used only as the centre of a progressive system, as at Baker Street/Marylebone Road in London.

The Effect of Randomness of Traffic on Intersection Delay

The two formulae for delay quoted above, (17) and (18), depend on the assumption of uniformity of flow. It is a fact of experience, however, that average delays at signals are longer than would be expected on the basis of these formulae at high levels of flow. This is easily explained, since random fluctuations in the number of vehicles arriving per cycle mean that on some occasions a surplus is left over to the next cycle, and delays are considerably increased. Although the average flow is less than the maximum and fluctuations are no more than random, it is quite possible for vehicles to be held for as many as four cycles.

It has been mentioned that a full mathematical treatment of random flow is difficult although, as Adams[1] has shown, it is possible to solve a large

Fig. 5

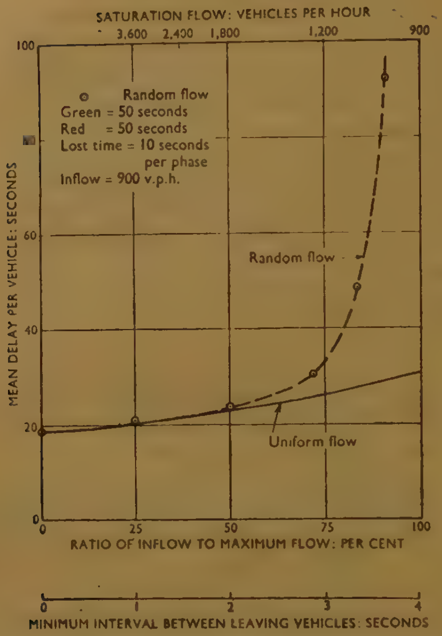


THEORETICAL VARIATION OF DELAY WITH FLOW, GIVEN UNIFORM FLOW, AT A TRAFFIC SIGNAL

variety of specific numerical problems. Greenshields[9] gives a step-by-step process for building up a solution, but admits that it is not practical. As an interim method, a partially theoretical method using random numbers could be very useful. As an indication of the results which could be obtained by this method, an investigation has been made in which random arrival times for 2,000 vehicles were obtained from random numbers. A fixed cycle was assumed, consisting of 100 seconds equally divided between green and red and with a lost time of 20 seconds. In order to simplify the computation the average delay was calculated for six values of $1/p$ (the interval between departing vehicles when the flow is at the saturation level). The results are shown in *Fig. 6* compared with the expected values on an assumption of uniform flow obtained from formula (14). The agreement is good up to the point which corresponds to 50 per cent of saturation, and even at 75 per cent of saturation the uniform-flow value is only about 20 per cent too low. But as the interval between departing vehicles, and hence the degree of saturation, increases further, the average delay increases more and more rapidly in the random case.

This example shows how delay is increased by increases in $1/p$, that is to say, by reductions in p, the saturation flow. Curves of a similar type would be expected if q, the flow, were to be increased while p remained fixed. Some examples of the average effect of changes in flow on London streets are shown in *Fig. 7*. These curves were obtained by combining the data from a number of controlled intersections.

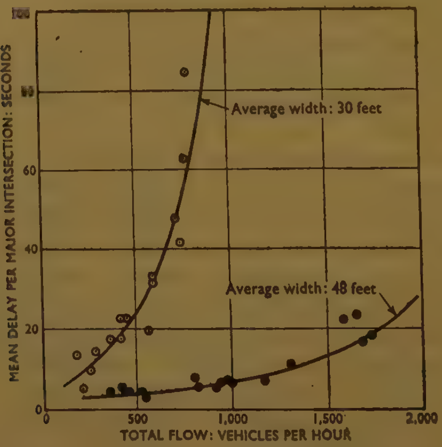
Fig. 6



Corrigendum:
for "900 v.p.h."
read "360 v.p.h."

Mean Delay at a Fixed-time Traffic Signal. Random and Uniform Flow

Fig. 7



Delay per Major Intersection on Two Groups of Streets in Central London

FORMATION OF QUEUES

A question related to that of delay is that of the rate at which a queue of vehicles will form when there is some obstruction, such as an intersection, a bottleneck, or an accident. Suppose that vehicles are arriving with flow q and space-mean speed v, and that once they have joined the queue their average flow is q_0 and their average speed v_0. Evidently if $q < q_0$, no queue forms, excluding temporary queues which oscillate in length. If $q > q_0$, a steadily increasing queue forms; let its length be x at time t, and suppose that $x = 0$ when $t = 0$. Consider the stretch of road of length x leading to the intersection. At $t = 0$ it contained kx vehicles, where $k = q/v$. At time t it contains k_0x vehicles, where $k_0 = q_0/v_0$, since it is filled with vehicles in the queue.

Now the difference between these two numbers equals the difference between the number entering the stretch and the number leaving it, $(q - q_0)t$. Hence $(k_0 - k)x = (q - q_0)t$.

If u is the rate at which the queue lengthens, then:

$$u = \frac{dx}{dt} = \frac{q - q_0}{k_0 - k}$$

that is to say:

$$u = \frac{q - q_0}{\frac{q_0}{v_0} - \frac{q}{v}} \quad \ldots\ldots\ldots\ldots \quad (19)$$

In the case of a signal-controlled intersection with effective red time $r + a$, cycle time c, and saturation flow p, it can be shown, by considering the distance travelled by a vehicle in one cycle, that the average speed in the queue is:

$$v_0 = \frac{1}{\frac{1}{v} + \frac{k_1(r + a)}{p(c - r - a)}} \quad \ldots\ldots \quad (20)$$

where k_1 denotes the concentration of stationary vehicles. The average flow of the queue is $q_0 = p(- r - a)/c$.

If there is a complete blockage, $q_0 = 0$ and $k_0 = k_1$ (the concentration of a stationary queue). Hence the queue lengthens at the rate:

$$u = \frac{q}{k_1 - \frac{q}{v}} \quad \ldots\ldots\ldots\ldots \quad (21)$$

As an example of the use of this formula, consider a single lane which is completely blocked, and in which vehicles form up with a headway of 18 feet. If the vehicles arrive with a speed of 15 miles per hour and a flow of 500 vehicles per hour, the queue increases at 1·9 miles per hour.

DISTRIBUTION OF TRAFFIC OVER ALTERNATIVE ROUTES

When the effect of some future improvement of a road system is to be judged, some estimate must be made of the distribution of traffic on the various roads affected, including not only new roads but all existing roads from which traffic may be diverted. This is usually done by making some rather arbitrary assumption about speeds on the new roads, and, given the results of an Origin and Destination survey, by assuming that every vehicle will travel by the quickest route. However, it has been seen that speed is a function of flow, so that redistribution of traffic upsets the pattern of speeds. The problem is to discover how traffic may be expected to distribute itself over alternative routes, and whether the distribution adopted is the most efficient one. Although there has not been a sufficiently detailed investigation of a road network to allow this to be done in practice, it seems worth while to consider the theoretical aspects of this problem.

Consider the case of a given flow of traffic Q, which has the choice of D alternative routes from a given origin to a given destination, numbered $1, 2, \ldots D$. Suppose that the flow Q is divided amongst them so that a quantity q_i follows route i. Then:

$$q_i \geqslant 0,\ i = 1, 2, \ldots D \text{ and } \sum_{i=1}^{D} q_i = Q$$

The problem is to determine the q_i subject to these conditions.

Suppose that the speed/flow relation is linear on each route in the range of flows considered, and that the flow on route i is q_i' before the addition of q_i. Then, if v_i is the speed on route i, $v_i = a_i - a_i'(q_i' + q_i)$ where a_i and a_i' are constants for route i, depending on street widths and intersections, etc. If l_i is the length of the route, the journey time is:

$$t_i = \frac{l_i}{a_i - a_i'q_i' - a_i'q_i}$$

This may be written in the simpler form:

$$t_i = \frac{b_i}{1 - \frac{q_i}{p_i}} \quad \ldots\ldots\ldots\ldots \quad (22)$$

where b_i and p_i are new constants depending on l_i, a_i, a_i' and q_i'. It is interesting to note the similarity of this expression to that for average delay at fixed-time signals (14).

Consider two alternative criteria based on these journey times which can be used to determine the distribution on the routes, as follows:

(1) The journey times on all the routes actually used are equal, and less than those which would be experienced by a single vehicle on any unused route.

(2) The average journey time is a minimum.

The first criterion is quite a likely one in practice, since it might be assumed that traffic will tend to settle down into an equilibrium situation in which no driver can reduce his journey time by choosing a new route. On the other hand, the second criterion is the most efficient in the sense that it minimizes the vehicle-hours spent on the journey. In practice, of course, drivers will be influenced by other factors, such as the state of the roads, and the comfort or discomfort of driving in general. However, it is clearly difficult to allow for these psychological factors.

Consider each criterion separately.

(1) *Equal Times*

The problem can be stated as follows: Given Q, b_1, $b_2, \ldots b_D$, p_1, $p_2, \ldots p_D$ and the relation (22), which routes must be used so that the journey time is the same on each, equal to t, say, and t is less than the journey time which would be experienced by a single vehicle on any route not used? Also, what is the value of t?

Suppose that $b_1 < b_2 < \ldots < b_D$. Any one of these quantities b_i is the journey time on route i when the additional flow $q_i = 0$. Suppose that $b_j < t \geqslant b_{j+1}$ for some route number j. Then clearly only the first j routes can be in use, for equation (22) shows that the journey time on any route i cannot be less than b_i. On the other hand, all of the first j routes must be in use; otherwise t would be greater than b_i for a route i which was not used. It follows that:

$$q_i = p_i\left(1 - \frac{b_i}{t}\right) \quad (\text{where } i = 1, 2, \ldots j). \quad . \quad . \quad (23)$$

these equations being obtained by re-writing (22). Summing over the first j routes gives:

$$Q = \sum_{i=1}^{j} p_i - \frac{1}{t}\sum_{i=1}^{j} p_i b_i \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

This equation gives the value of Q for which t is the appropriate journey time. If Q is calculated for each value of t, it is possible to find the solution to the problem by picking out the value of t which corresponds to the given Q.

(2) *Minimum Average Time*

In this case the problem is as follows:—Given Q, b_1, $b_2, \ldots b_D$, p_1, $p_2, \ldots p_D$ as before, find q_1, $q_2 \ldots q_D$ satisfying the conditions such that the average journey time T is a minimum.

Now $$T = \sum_{i=1}^{D} q_i t_i / Q \qquad \text{(25}$$

Note that if z_i is the average number of additional vehicles on route i at an instant, then $z_i = q_i t_i$. Let $Z = \sum_{i=1}^{D} z_i$ be the total number of additional vehicles on the routes.

Then $$T = Z/Q \qquad (26)$$

Now the criterion considered is that T should be a minimum. Since Q is constant, this is equivalent to Z—the total number of additional vehicles *en route* being a minimum.

Suppose that the solution has an additional flow q_i on route i, $i = 1, 2, \ldots D$. Then if i and j are any two routes in use, the value of Z must not be altered by the transfer of an infinitesimal increment of flow δq from route i to route j. This means that:

$$\frac{d(q_i t_i)}{dq_i} = \frac{d(q_j t_j)}{dq_j} \qquad (27)$$

Hence $$\frac{d(q_i t_i)}{dq_i} = \epsilon \qquad (28)$$

for all the routes in use, where ϵ is a constant independent of the route number.

A route j will not be used if $\left[\frac{d(q_j t_j)}{dq_j}\right]_{q=0} \geqslant \epsilon$, for in that case, transfer to it of an increment δq from any route in use will increase the value of Z.

Now $$\frac{d(q_i t_i)}{dq_i} = \frac{b_i}{\left(1 - \frac{q_i}{p_i}\right)^2} \qquad (29)$$

and so $$\left[\frac{d(q_i t_i)}{dq_i}\right]_{q=0} = b_i \qquad (30)$$

Thus if $b_j < \epsilon < b_{j+1}$ only the first j routes will be in use.

Hence $$1 - \frac{q_i}{p_i} = \sqrt{\frac{b_i}{\epsilon}}$$

and so $$p_i - q_i = p_i \sqrt{\frac{b_i}{\epsilon}} \quad \text{where } i = 1, 2, \ldots j.$$

Summing over the first j routes gives:

$$\sum_{i=1}^{j} p_i - Q = \sum_{i=1}^{j} p_i \sqrt{\frac{b_i}{\epsilon}} \qquad (31)$$

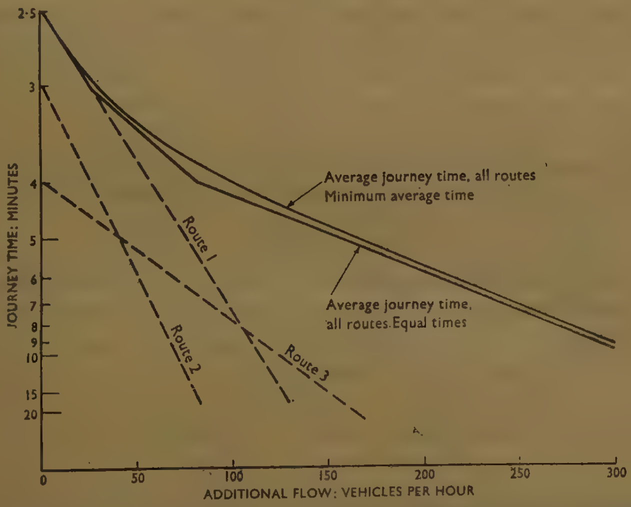
With a few lines of algebra it can be shown that :

$$T = \frac{1}{Q}\left\{ \frac{\left(\sum_{i=1}^{j} p_i\sqrt{b_i}\right)^2}{\sum_{i=1}^{j} p_i - Q} - \sum_{i=1}^{j} p_i b_i \right\} \quad . \; . \; . \; . \; . \qquad (32)$$

and the distribution of flows is given by :

$$\left.\begin{aligned} q_h &= p_h\left\{1 - \frac{\left(\sum_{i=1}^{j} p_i - Q\right)\sqrt{b_h}}{\sum_{i=1}^{j} p_i b_i}\right\}, \; h = 1, 2, \ldots j \\ q_h &= O, \qquad\qquad\qquad\qquad h = j+1, \ldots D. \end{aligned}\right\} \quad . \qquad (33)$$

Fig. 8



Distribution of Traffic on Alternative Routes Journey-Time/Flow Relations for a Hypothetical Example with Three Alternatives

The result of calculating the average journey time by each method for a particular hypothetical example is illustrated in *Fig.* 8. Here the average journey time is plotted, on an inverse scale, against the additional flow using some or all of the three routes. The inverse scale of journey time is used so that the individual relations between journey time and additional flow, also shown in the Figure, are linear. It will be seen that the advantage given by the minimum average method is not very great in this case.

Flows on a Network

In the case of a network of roads the theoretical problem becomes very complicated. However, it is possible to suggest a general line of approach. The same two criteria may be used as alternatives. That is to say, alternative routes between any two points may be chosen so that either all routes used have the same journey time, or else the average journey time for all journeys, and therefore the average number of vehicles on the network, is the minimum.

" BEFORE-AND-AFTER " STUDIES

Almost every traffic investigation involving a study of actual conditions is a " before-and-after " problem. Even general surveys and censuses are normally undertaken to discover trends in traffic conditions, which means making one comparison or a whole series of comparisons. Consider two sets of observations, one taken before and the other after a change. It is assumed that each " before " observation is chosen at random from a very large number of possible values (the population), and each choice is made independently of the others. The set of observations is then a random sample from the population. Similarly the " after " observations are assumed to form a random sample from a very large population, in general differing from the first population. The concepts of a random sample from a population and of significance, which is discussed below, are dealt with fully in most books on statistics, for example, Yule and Kendall.[10]

Suppose that the number of observations (the sample size) is n " before " and N " after," and the observations, which may be measurements of journey time, delay, speed, parking concentration or any other measurable quantity, are $x_1, x_2, \ldots x_n$ " before " and $X_1, X_2, \ldots X_N$ " after." The standard practice is to calculate the means :

$$\bar{x} = \sum_{i=1}^{n} x_i/n \quad \text{and} \quad \bar{X} = \sum_{j=1}^{N} X_j/N$$

and to consider the difference between them, $d = \bar{X} - \bar{x}$, as an index of the change in the quantity measured. Now owing to purely random fluctuations, even if there is no difference between the populations, the means $\bar{x}$ and $\bar{X}$ may be expected to differ. In order to test whether the difference is likely to mean a " true " difference between the populations, it is necessary to consider the expected variation in x and $\bar{X}$. Suppose that the population means are m and M respectively, and that $\delta = M - m$. An estimate s of the standard deviation σ_x of the population of x's is calculated from the " before " sample ; similarly an estimate S of the standard deviation σ_X of the population of X's is found from the " after " sample. These two estimates are defined as follows :

$$\left.\begin{aligned} s &= \sqrt{\left\{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}\right\}} \\ S &= \sqrt{\left\{\frac{\sum_{j=1}^{N}(X_j - \bar{X})^2}{N-1}\right\}} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad (34)$$

In order to test the significance of $d = \bar{X} - \bar{x}$, the estimated standard deviation of this quantity is calculated. This is :

$$s(d) = \sqrt{\left\{\frac{s^2}{n} + \frac{S^2}{N}\right\}}$$

that is to say, $$s(d) = \sqrt{\left\{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n(n-1)} + \frac{\sum_{j=1}^{N}(X_j - \bar{X})^2}{N(N-1)}\right\}} \quad . \quad (35)$$

Now if the samples are of reasonable size and the populations have approximately " normal " distributions, which is often the case, the ratio $d/s(d)$ is also approximately a normal variable with unit standard deviation. If there is no difference between population means, that is to say, if $\delta = 0$, $d/s(d)$ also has an expected value of zero. Tables of a normal variable with zero mean and unit standard deviation are given in most books on statistics; a very complete Table is available.[11] The value found in the experiment can be looked up in such a Table and the probability that a difference at least as large (either positive or negative) would occur by chance can then be read off. If this probability (α) is less than 0·05 the difference d is said to be significantly different from zero (often abbreviated to " significant ") at the 5-per-cent level ; if it is less than 0·01, d is significant at the 1-per-cent level, and so on. The corresponding values of $d/s(d)$ are 1·96 and 2·58. The 5-per-cent level is the one most commonly used ; if used consistently in a long series of " before-and-after " studies it would mean that in all cases where there was no " true " difference the sample difference would be wrongly accounted as significant about once in 20 times.

Size of Sample

The size of sample in each case may be determined by the periods during which observations can be made, or, more likely, by the man-hours which can be spared. On other occasions the " before " study may be limited, with no possibility of extending the sample because an irrevocable change has been made, while the " after " sample can be much larger. It is useful here to consider the effect of increasing N, if n is fixed, on $s(d)$, the standard error of the difference, assuming that $\sigma_x = \sigma_X$. Expressed as a percentage of the value when $n = N$, the results are as follows :

Ratio of sample sizes ($N{:}n$)	Standard error of difference
1:1	100
2:1	87
4:1	79
10:1	74
100:1	71

Evidently it is not worth while to increase the " after " sample to more than 4 times the " before " sample, unless the observations are very easy to make.

Where the whole experiment is planned from the outset, and there is no limit to the sample sizes, the most efficient method, assuming that little difference between the population standard deviations is expected, is to have equal samples sizes, say n, before and after. It is clear from the form of the equation for the standard error of the difference that the greater the sample size the greater the precision of d. On the other hand the smallest sample means the least work. Some criterion is needed to decide the minimum sample size which is sufficient.

Suppose that the smallest difference of any concern is δ and that the significance level α is to be 5 per cent. If there is a real population difference equal to δ, there is still a possibility that the difference between a pair of samples will be rejected as non-significant. Suppose that the experimenter decides that he is willing to accept a 10-per-cent risk of making this mistake. Now in this case the probability that $-1{\cdot}96 < d/s(d) < 1{\cdot}96$, and therefore the sample difference is non-significant, is the same as the probability that :

$$-1{\cdot}96-\frac{\delta}{s(d)}<\frac{d-\delta}{s(d)}<1{\cdot}96-\frac{\delta}{s(d)}$$

and this is to be 0·1. If δ is such that $1{\cdot}96-\dfrac{\delta}{s(d)}=-1{\cdot}28$, that is to say, if $\dfrac{\delta}{s(d)}=3{\cdot}24$, the probability of the left-hand inequality is approximately 0·1, since $\dfrac{d-\delta}{s(d)}$ is approximately a normal variable * with zero mean and unit standard deviation. In this case the probability that $\dfrac{d-\delta}{s(d)}<-1{\cdot}96-\dfrac{\delta}{s(d)}=-5{\cdot}20$ is negligible so that the condition required is

$$\frac{\delta}{s(d)}=3{\cdot}24$$

---

* Strictly $\dfrac{d-\delta}{s(d)}$ has a " t-distribution ", and the t-test should be used, but the error in assuming normality is small when sample sizes of 100 or more are involved and this is usually the case in traffic studies.

In terms of the sample size and the standard error of a single observation, σ, assumed to be the same before and after, this gives

$$\frac{\delta}{\sigma\sqrt{\frac{2}{n}}} = 3{\cdot}24$$

or

$$n = 21{\cdot}0\left(\frac{\sigma}{\delta}\right)^2$$

In general, the sample size is

$$n = \gamma\left(\frac{\sigma}{\delta}\right)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad (36)$$

where γ depends on the significance level α and the acceptable probability β of rejecting a true difference δ. Values of γ for some typical values of α and β are given in Table 2. In order to use this Table some estimate of the standard error of a single variable is required. Generally previous observations provide this; otherwise a guess must be made, and the sample size corrected, if necessary, in the light of the first results.

TABLE 2.—VALUES OF γ IN FORMULA $n = \gamma\left(\frac{\sigma}{\delta}\right)^2$ FOR ASSESSING SAMPLE SIZE

Probability of rejecting "true" difference (β): per cent	Significance level, α: per cent		
	5	2	1
10	21·0	26·0	29·8
5	26·0	31·5	35·6
2	32·2	38·4	42·9
1	36·7	43·3	48·1

As an example, consider a "before-and-after" study of journey times. The ratio of the standard deviation to the mean is often about 1:4, so that if the significance level is 5 per cent, and the probability of rejecting a 10-per-cent difference is also to be 5 per cent, the sample size is

$$n = 26{\cdot}0\left(\frac{\sigma}{\delta}\right)^2 = 26{\cdot}0(2{\cdot}5)^2 = 160 \text{ approximately.}$$

That is, samples of about 160 runs before and after are required.

Several Routes and Several Periods

It is often necessary to consider the effect of a change in road lay-out or method of control on journey time on a number of routes. For instance, the whole effect of a one-way system can only be found by studying every route affected by the system. Similarly it may be necessary to cover different periods in both the "before" and the "after" experiment, because the effect which is sought varies with time of day, day of week, or season of year. Methods of combining results for several routes and several periods are discussed in Appendix VI.

Average Journey Time, Distance, and Speed

In summarizing a "before-and-after" study it is often necessary to give comparative figures for journey time, distance, and speed, averaged over all periods and times. This is particularly desirable in the case of one-way systems. Here the average speed on a journey usually increases, but the average distance travelled per journey also increases, owing to the less direct routes followed. If the average speed increases by a greater proportion than the average distance, the average journey time is reduced, and there is a saving in vehicle-hours. On the other hand the increase in speed may be proportionally less than that of distance, so that the average journey time is longer and vehicle-hours are wasted. The appropriate average values to reveal this effect are given below.

Suppose that the various quantities measured for a typical route during a typical period are expressed symbolically as follows:

Duration of period		Length of route	Flow	Mean journey time
θ	Before	l	q	t
	After	L	Q	T

Then it is shown in Appendix VI that the overall weighted average journey time "before" is:

$$\bar{t}_w = \Sigma(q + Q)\theta t/\Sigma(q + Q)\theta \quad . \; . \; . \; . \; . \; . \; . \quad (37)$$

where the summations extend over all routes and periods.

The average distance per journey "before," weighted in the same way is:

$$\bar{l}_w = \Sigma(q + Q)\theta l/\Sigma(q + Q)\theta \quad . \; . \; . \; . \; . \; . \; . \quad (38)$$

The average space-mean speed "before" is:

$$\bar{v}_s = \bar{l}_w/\bar{t}_w = \Sigma(q + Q)\theta l/\Sigma(q + Q)\theta t \quad . \; . \; . \quad (39)$$

The corresponding quantities for the "after" results are obtained by replacing lower case by capital letters in these expressions, the term $q + Q$ remaining unaltered.

Two examples of results for one-way systems treated in this way are shown in Table 3.

It will be seen that in the first case the increase in distance outweighs the increase in speed, and the average journey time is increased; in the second case, however, the speed change dominates and the journey time is reduced.

TABLE 3.—AVERAGE EFFECTS OF TWO ONE-WAY SYSTEMS

Site of one-way system		Average distance per journey: miles	Average journey speed: m.p.h.	Average journey time: minutes
Slough	Before	0·43	13·8	1·88
	After	0·52	15·2	2·06
	Change: per cent	+ 21	+ 10	+ 10
Holland Road (London)	Before	0·42	13·2	1·93
	After	0·46	15·9	1·73
	Change: per cent	+ 10	+ 20	− 10

Vehicle-Mileage and Vehicle-Hours

It should be noted that the quantities which appear as denominators and numerators in the expressions for average journey time, distance, and speed all have a direct physical meaning. It is pointed out in Appendix VI that, for the typical route and period, $\frac{1}{2}(q + Q)$ is taken as an estimate of the average flow, assumed to be the same before and after. It follows that, for the "before" half of the experiment, for instance:

$\frac{1}{2}\Sigma(q + Q)\theta$ = total vehicles using routes during periods studied;

$\frac{1}{2}\Sigma(q + Q)\theta l$ = total vehicle-mileage on routes during periods studied;

and $\frac{1}{2}\Sigma(q + Q)\theta t$ = total vehicle-hours on routes during periods studied.

Hence: $$\text{mean length of journey} = \frac{\text{vehicle-miles}}{\text{vehicles}}$$

$$\text{mean journey time} = \frac{\text{vehicle-hours}}{\text{vehicles}}$$

and $$\text{space-mean speed} = \frac{\text{vehicle-miles}}{\text{vehicle-hours}}.$$

As was pointed out earlier, if very sensitive statistical tests are required it may be better to apply them to the averages of the speeds of individual vehicles rather than to average journey times. However, the resulting quantities have no direct physical meaning, and should not be used to predict changes in vehicle-hours. The correct course is to use speeds, if necessary, for statistical tests, but journey times in all cases for assessing gains or losses.

FUTURE WORK

This Paper has presented a number of theoretical considerations and formulae which are relevant to the practical problems arising in traffic engineering research. A great deal more work is needed, both in the theoretical field and in the application of these and similar methods in actual cases. Many of the results given can easily be extended to more complicated situations, but this is not always possible. More work is required to find the theoretical relation between delay and flow at intersections, particularly those with vehicle-actuated signals, and the effect of linking signal installations, and to verify any results obtained. Another question of importance is that of the effect of the high correlation between the journey times of two vehicles following a given route within a very short interval; this means, for example, that a sample of journey times of successive vehicles has not the same value as a sample of equal size taken at intervals of, say, 10 minutes.

CONCLUSION

It has been demonstrated that many traffic problems involve theoretical considerations and that a knowledge of elementary statistics is desirable for work on traffic research. It is hoped that the results and ideas given in this Paper will be helpful to road engineers when dealing with the types of investigation discussed, and will stimulate further research on the subject.

ACKNOWLEDGEMENT

The investigations described were carried out as part of the programme of the Road Research Board of the Department of Scientific and Industrial Research. The Paper is presented by permission of the Director of Road Research.

REFERENCES

1. W. F. Adams, " Road Traffic Considered as a Random Series." J. Instn Civ. Engrs, vol. 4, p. 121 (November 1936).
2. W. H. Glanville, " The Study of Road Traffic." Jt Summ. Mtg, Instn Civ. Engrs and Instn Civ. Engrs, I., 1949.
3. W. H. Glanville, " Road Safety and Road Research." J. Roy. Soc. Arts, vol. 99, p. 114 (Jan. 1951).
4. R. J. Smeed and G. T. Bennett, " Research on Road Safety and Traffic Flow." Road Paper No. 29, Instn Civ. Engrs, 1949.
5. O. K. Normann and W. P. Walker, " Highway Capacity : Practical Applications of Research." *Publ. Rds*, Wash., vol. 25, p. 201 (No. 10, 1949); p. 237 (No. 11, 1949).

6. John P. Kinzer, "Application of the Theory of Probability to Problems of Highway Traffic." *Abstract in* Proc. Inst. Traffic E., Fifth Annual Meeting (Oct. 2–3, 1934), p. 118.
7. A. J. H. Clayton, "Road Traffic Calculations." J. Instn Civ. Engrs, vol. 16, p. 247 (June 1941).
8. F. Garwood, "An application of the theory of probability to the operation of vehicular-controlled traffic signals." Suppl. J. Roy. Stat. Soc., vol. VII, p. 65 (No. 1, 1940–41).
9. B. D. Greenshields, D. Schapiro, and E. L. Ericksen, "Traffic Performance at Urban Street Intersections." Yale Univ. Bur. of High. Traff., Tech. Rep. No. 1. New Haven, Conn., 1947.
10. G. U. Yule and M. G. Kendall, "An Introduction to the Theory of Statistics" (14th edition). Griffin (London), 1950.
11. K. Pearson (ed.), "Tables for Statisticians and Biometricians: Part I." Biometrika Off., Univ. Coll., London.

The Paper is accompanied by eight sheets of drawings, from which the Figures in the text have been prepared, and by six Appendices.

APPENDIX I

GLOSSARY AND SYMBOLS

Headway.—The distance from the front of a vehicle to the front of the one directly ahead.

Flow.—The number of vehicles passing a given point on the road per unit time, in one direction or both according to the context. (q).

Saturation flow (at traffic signals).—The flow which is reached when vehicles are travelling at minimum achievable headway. (p).

Speed.—(v).

Journey speed.—The average speed on a journey, including stops.

Running-speed.—The average speed while in motion. (v_r).

Concentration.—The number of vehicles per unit length of road. (k).

Time-distribution of speed.—The frequency distribution of speeds of vehicles in time as they pass a point on the road. The subscript t is used for the time-distribution.

Space-distribution of speed.—The frequency distribution of speeds of vehicles on a uniform road at an instant of time. The subscript s is used for the space-distribution.

Time-mean speed.—The mean of the time-distribution of speed. $(\bar{v}_t)$.

Space-mean speed.—The mean of the space-distribution of speed. $(\bar{v}_s)$.

"*Green*" *time* (at traffic signals).—The time during which the signal for a particular phase is green or amber. (g).

"*Red*" *time* (at traffic signals).—The time during which the signal for a particular phase is red or red/amber. (r).

"*Lost*" *time* (at traffic signals).—The part of the "green" time for a particular phase which is effectively "lost", chiefly due to acceleration delay. (a). It is approximately true that no vehicles leave the intersection at the beginning of the "green" time until the lost time has elapsed, after which they leave at the saturation flow.

Effective red time (at traffic signals).—The "red" time plus the "lost" time.

Delay (at traffic signals).—The difference between the average journey time through the intersection and the time for a run which is not stopped or slowed down by the signals. (Average delay : t).

Journey time.—The time on a journey including stops.

Note. The symbols shown above may appear in the text in a modified form, for example, with subscripts or as capitals, with meanings which are defined in the context.

APPENDIX II

RELATION BETWEEN TIME- AND SPACE-MEAN SPEEDS

The time-mean speed is defined by equation (2) as follows :

$$\bar{v}_t = \sum_{i=1}^{C} q_i v_i / Q$$

where q_i denotes the flow of the subsidiary stream whose speed is v_i and Q denotes the total flow. The space-mean speed is given by equation (3) as :

$$\bar{v}_s = \sum_{i=1}^{C} k_i v_i / K$$

where k_i denotes the concentration of the subsidiary stream whose speed is v_i and K denotes the total concentration. Substituting for the value of q_i (as given in equation (4)) in the expression for $\bar{v}_t$ gives

$$\begin{aligned}
\bar{v}_t &= \sum_{i=1}^{C} k_i v_i^2 / Q \\
&= K \sum_{i=1}^{C} f_i' v_i^2 / Q \text{ by the definition of } f_i' \\
&= \sum_{i=1}^{C} f_i' v_i^2 / \bar{v}_s \text{ from (5)} \\
&= \sum_{i=1}^{C} f_i' \{\bar{v}_s + (v_i - \bar{v}_s)\}^2 / \bar{v}_s \\
&= \left\{ \left(\sum_{i=1}^{C} f_i' \right) \bar{v}_s^2 + \sum_{i=1}^{C} f_i' (v_i - \bar{v}_s)^2 \right\} / \bar{v}_s \\
&\left[\text{since } \sum_{i=1}^{C} f_i' (v_i - \bar{v}_s) = 0 \right] \\
&= \bar{v}_s + \frac{\sigma_s^2}{\bar{v}_s},
\end{aligned}$$

where $\sigma_s = \sqrt{\left\{ \sum_{i=1}^{C} f_i' (v_i - \bar{v}_s)^2 \right\}}$ is the standard deviation of the space distribution of speed. This relation shows that $\bar{v}_t \geqslant \bar{v}_s$ on all occasions, and if there is any variation in speed at all $\sigma_s > 0$ and $\bar{v}_t > \bar{v}_s$.

APPENDIX III

MINIMUM CYCLE AND MEAN DELAY FOR A FIXED-TIME TRAFFIC SIGNAL, ASSUMING UNIFORM FLOW

The formulae given below have been derived in substantially the same form by other authors. In particular the method of derivation follows that of Clayton [7], but it is given here for the sake of completeness.

The symbols used are defined on p. 15. The cycle is minimum when each green period is saturated, that is to say, when vehicles leave at minimum headway up to the end of the period but no vehicles are held over to the next cycle. In phase 1, the period a_1 is lost, so that the number of vehicles cleared is $p_1(g_1 - a_1)$. But this must equal the number arriving during one cycle, q_1c. Hence:

$$p_1(g_1 - a_1) = q_1c,$$

that is to say,
$$g_1 = a_1 + \frac{q_1}{p_1}c.$$

Similarly,
$$g_2 = a_2 + \frac{q_2}{p_2}c.$$

Adding these two last equations gives:

$$c = A + \left(\frac{q_1}{p_1} + \frac{q_2}{p_2}\right)c,$$

that is to say,
$$c = \frac{A}{1 - \frac{q_1}{p_1} - \frac{q_2}{p_2}} \quad \text{(Adams' Formula).}$$

In the general case, if the cycle exceeds this minimum the condition must be satisfied that the number of vehicles which arrive during one cycle is less than the maximum number which could be cleared (that is to say, if the green period was saturated for each phase). That is:

$$q_1c \leqslant p_1(g_1 - a_1) \text{ and } q_2c \leqslant p_2(g_2 - a_2)$$

These conditions provide upper limits to the two flows.

Consider the delay to successive vehicles, remembering that the flow is assumed to be uniform. *Fig. 4* shows that successive vehicles leave the intersections at times $a, a + \frac{1}{p}, a + \frac{2}{p}, \ldots$ after the appearance of the green, until the period of saturation flow ceases. Since vehicles arrive at intervals $1/q$, the first vehicle to be stopped would have cleared the intersection at any time between 0 and $1/q$ after the beginning of the red period if it had not been stopped. Suppose that it actually arrives at such a time that it would have cleared the intersection at the average time $1/2q$ after the beginning of the red, as shown in *Fig. 4*. Then it is delayed by an amount $r + a - \frac{1}{2q}$. The next vehicle arrives an interval $1/q$ later, and leaves $1/p$ later, than the first, so that its delay is $\frac{1}{q} - \frac{1}{p}$ less. Similarly each successive vehicle is delayed by $\frac{1}{q} - \frac{1}{p}$ less than the previous one. Suppose that the average delay to the last delayed vehicle is $\frac{1}{2}\left(\frac{1}{q} - \frac{1}{p}\right)$; this is reasonable, since it is equally likely to be anywhere between 0 and $\frac{1}{q} - \frac{1}{p}$. Suppose also that the number of vehicles delayed is n. The average delay to delayed vehicles is the average of the delays to the first and the last. Evidently the first vehicle is delayed by an amount $\frac{2n-1}{2}\left(\frac{1}{q} - \frac{1}{p}\right)$ so that the average delay to delayed vehicles is $\frac{1}{2}n\left(\frac{1}{q} - \frac{1}{p}\right)$. But it has already been shown that the average delay to the first vehicle can be expressed as $r + a - \frac{1}{2q}$, and equating the two expressions for this quantity gives:

$$n = \left(r + a - \frac{1}{2p}\right) \Big/ \left(\frac{1}{q} - \frac{1}{p}\right)$$

The total delay is $\frac{1}{2}n^2\left(\frac{1}{q} - \frac{1}{p}\right)$ and the average for all vehicles is:

$$t = n^2\left(\frac{1}{q} - \frac{1}{p}\right) \Big/ qc$$

that is to say,
$$t = \frac{\left(r + a - \frac{1}{2p}\right)^2}{2c\left(1 - \frac{q}{p}\right)}$$

Formulae differing slightly from this can be obtained by altering the assumptions, but when evaluated the differences are generally trivial.

APPENDIX IV

Optimum Phase Times for a Fixed-Time Two-Phase Traffic Signal

The average delay for the intersection as a whole is given by equation (16), which reads:

$$T = \frac{1}{2Qc}\left\{\frac{q_1(r_1 + a_1)^2}{1 - \frac{q_1}{p_1}} + \frac{q_2(r_2 + a_2)^2}{1 - \frac{q_2}{p_2}}\right\}$$

Let $r_1 + a_1 = xc$

and $r_2 + a_2 = yc.$

Addition gives $c + a = (x + y)c,$

therefore $c = \frac{\text{[illegible]}}{x + y - 1}.$

In terms of x and y,
$$T = \frac{A}{2Q(x + y - 1)}\left\{\frac{q_1x^2}{1 - \frac{q_1}{p_1}} + \frac{q_2y^2}{1 - \frac{q_2}{p_2}}\right\}$$

The conditions given on p. 357 which ensure that vehicles do not accumulate indefinitely on either phase, become:

$$x \leqslant \xi < 1$$

and $y \leqslant \eta < 1,$

where $\xi = 1 - \frac{q_1}{p_1}$

and $\eta = 1 - \frac{q_1}{p_1}$

Let $\lambda = Aq_1/2Q\xi$

and $\mu = Aq_2/2Q\eta$

then $T = (\lambda x^2 + \mu y^2)/(x + y - 1)$

Now it is generally assumed that the optimum cycle is the shortest one. The shortest cycle occurs when x and y have their maximum values, that is to say, when $x = \xi$ and $y = \eta$. In order to test whether t is a minimum at this point, consider the values of $\frac{\partial T}{\partial x}$ and $\frac{\partial T}{\partial y}$ when $x = \xi$ and $y = \eta$.

$$\left[\frac{\partial T}{\partial x}\right]_{\xi,\eta} = \frac{\lambda\xi^2 - \mu\eta^2 - 2\lambda\xi(1 - \eta)}{(\xi + \eta - 1)^2}$$

and
$$\left[\frac{\partial T}{\partial y}\right]_{\xi,\eta} = \frac{\mu\eta^2 - \lambda\xi^2 - 2\mu\eta(1 - \xi)}{(\xi + \eta - 1)^2}$$

Now the only possible changes in x and y in this case are reductions. Hence T is minimum when $x = \xi$, $y = \eta$, if $\frac{\partial T}{\partial x} < 0, \frac{\partial T}{\partial y} < 0$

Now

$$(\xi + \eta - 1)^2\left[\frac{\partial T}{\partial y}\right]_{\xi,\eta} = \mu(\eta^2 - 2\eta + 2\xi\eta) - \lambda\xi^2$$
$$= -\mu\{2\eta(1 - \eta) + (\xi + \eta)^2\} - (\lambda - \mu)\xi^2$$

Suppose that $\lambda \geqslant \mu$. If this condition is not satisfied, phases 1 and 2 can be interchanged, and it will then be satisfied. Then:

$$\left[\frac{\partial T}{\partial y}\right]_{\xi,\eta} < 0$$

since the right-hand side of the above expression is negative while $(\xi + \eta - 1)^2$ is positive. In this case, therefore, the condition that T is the minimum is:

$$\left[\frac{\partial T}{\partial x}\right]_{\xi,\eta} < 0$$

that is to say,

$$\lambda\xi^2 - \mu\eta^2 - 2\lambda\xi(1 - \eta) < 0$$

or

$$(\mu/\lambda)\eta^2 + 2\xi(1 - \eta) - \xi^2 > 0$$

If, however, $(\mu/\lambda)\eta^2 + 2\xi(1 - \eta) - \xi^2 < 0$, T is a minimum at (x, η), where x is given by $\left[\frac{\partial T}{\partial x}\right]_{\xi,\eta} = 0$,

that is to say,

$$\lambda x^2 - \mu\eta^2 - 2\lambda x(1 - \eta) = 0$$

The root of this equation, in the range $0 < x < \eta$, is:

$$1 - \eta + \sqrt{\{(1 - \eta)^2 + (\mu/\lambda)\eta^2\}}$$

In terms of the original quantities, the condition $\lambda \geqslant \mu$ becomes:

$$\left(\frac{1}{q_1} - \frac{1}{p_1}\right) \geqslant \left(\frac{1}{q_2} - \frac{1}{p_2}\right)$$

This determines which phase shall be called 1 and which 2. The condition that the minimum cycle shall be the optimum is

$$q_1\left(1 - \frac{q_1}{p_1}\right) - q_2\left(1 - \frac{q_2}{p_2}\right) < \frac{2q_1q_2}{p_2}.$$

If this condition is not satisfied, the minimum value of T occurs when

$$c = \frac{A}{\sqrt{\{(1 - \eta)^2 + (\mu/\lambda)\eta^2\}}}$$
$$= \frac{A}{\sqrt{\left\{\left(\frac{q_2}{p_2}\right)^2 + \frac{q_2}{q_1}\left(1 - \frac{q_1}{p_1}\right)\left(1 - \frac{q_2}{p_2}\right)\right\}}}$$

APPENDIX V

Delay for an "Ideal" Vehicle-Actuated Traffic Signal

Here the cycle is assumed to be the minimum, so that:

$$c = \frac{A}{1 - \frac{q_1}{p_1} - \frac{q_2}{p_2}}$$

$$r_1 + a_1 = \left(1 - \frac{q_1}{p_1}\right)c$$

and

$$r_2 + a^2 = \left(1 - \frac{q_2}{p_2}\right)c$$

Substituting these values in equations (14) and (16) gives the following equations for average delays t_1 and t_2 on each phase and T for the intersection as a whole :

$$t_1 = \tfrac{1}{2}c\left(1 - \frac{q_1}{p_1}\right) = \tfrac{1}{2}A\left(1 - \frac{q_1}{p_1}\right)\bigg/\left(1 - \frac{q_1}{p_1} - \frac{q_2}{p_2}\right)$$

$$t_2 = \tfrac{1}{2}c\left(1 - \frac{q_2}{p_2}\right) = \tfrac{1}{2}A\left(1 - \frac{q_2}{p_2}\right)\bigg/\left(1 - \frac{q_1}{p_1} - \frac{q_2}{p_2}\right)$$

and
$$T = \frac{A}{2Q\left(1 - \frac{q_1}{p_1} - \frac{q_2}{p_2}\right)}\left\{q_1\left(1 - \frac{q_1}{p_1}\right) + q_2\left(1 - \frac{q_2}{p_2}\right)\right\}$$

In the particular case where the intersection is fully symmetrical, so that $q_1 = q_2 = q$, $p_1 = p_2 = p$, and $a_1 = a_2 = a$, the average delay on each phase is:

$$t = \frac{a\left(1 - \frac{q}{p}\right)}{1 - \frac{2q}{p}}$$

APPENDIX VI

COMPARISON OF JOURNEY TIMES IN "BEFORE-AND-AFTER" STUDIES WITH SEVERAL ROUTES AND WITH SEVERAL PERIODS

Several Routes

Suppose there are E routes, and the results of the experiment are symbolically as follows :

Route		Flow	Sample size	Mean journey time	Standard deviation of journey times
i	Before	q_i	n_i	t_i	s_i
	After	Q_i	N_i	T_i	S_i

It is desired to combine the "before" results and compare them with the "after" results. A simple method is to find the arithmetic mean of the mean journey times on the routes, that is to say :

$$\bar{t} = \sum_{i=1}^{E} t_i/E, \quad \bar{T} = \sum_{i=1}^{E} T_i/E$$

The standard error of $\bar{T} - \bar{t}$ is then :

$$s(\bar{T} - \bar{t}) = \frac{1}{E}\sqrt{\left\{\sum_{i=1}^{E}\left(\frac{s_i^2}{n_i} + \frac{S_i^2}{N_i}\right)\right\}}$$

and the usual test for significance can be applied.

However, if the flows on the various routes are very different, it is better to attach weights to the journey times to allow for this. If the experiment has been properly planned, the difference between the measured flows on any route i before and after will not be significant. Otherwise it must be assumed that there is a real difference in the flow, and this would itself be expected to affect journey times and spoil the experiment. The two flows can therefore be pooled to give an average flow of $\tfrac{1}{2}(q_i + Q_i)$. These average flows can then be used as weights, giving weighted average journey times :

$$\bar{t}_w = \sum_{i=1}^{E} (q_i + Q_i)t_i \Big/ \sum_{i=1}^{E} (q_i + Q_i)$$

$$\bar{T}_w = \sum_{i=1}^{E} (q_i + Q_i)T_i \Big/ \sum_{i=1}^{E} (q_i + Q_i)$$

The factor $\frac{1}{2}$ cancels out between denominator and numerator. It is essential for a fair comparison that the weights used should be the same before and after. The standard error of $\bar{T}_w - \bar{t}_w$ is:

$$s(\bar{T}_w - \bar{t}_w) = \frac{1}{\sum_{i=1}^{E} (q_i + Q_i)} \sqrt{\left\{ \sum_{i=1}^{E} (q_i + Q_i)^2 \left(\frac{s_i^2}{n_i} + \frac{S_i^2}{N_i} \right) \right\}}$$

and this is used for testing the significance of $\bar{T}_w - \bar{t}_w$.

Several Periods

Another common case occurs when journey times on a single route are studied at different times of the day and/or on different days of the week. Suppose that there are F periods, each period being distinguished from the others because of differences in conditions. It is essential to cover the same periods (that is to say, the same hours of the same days of the week) "before" and "after." Suppose that in each period a sample number of runs is taken, which can be regarded as representative of the period as a whole. Let the results be symbolically as follows:

Period	Duration of period		Flow	Sample size	Mean journey time	Standard deviation of journey times
j	θ_j	Before	q_j	n_j	t_j	s_j
		After	Q_j	N_j	T_j	S_j

As before, the arithmetic mean of the journey times may be used, but again it is more reasonable to introduce weights. Here, since both the flows and the periods vary, it is appropriate to weight by the number of vehicles concerned in each period. Using the average flow as before, this is $\frac{1}{2}(q_j + Q_j)\theta_j$ for period j. The corresponding weighted mean journey times are:

$$\bar{t}_w = \sum_{j=1}^{F} (q_j + Q_j)\theta_j t_j \Big/ \sum_{j=1}^{F} (q_j + Q_j)\theta_j$$

and $$\bar{T}_w = \sum_{j=1}^{F} (q_j + Q_j)\theta_j T_j \Big/ \sum_{j=1}^{F} (q_j + Q_j)\theta_j$$

and the standard error of the difference is:

$$s(T_w - t_w) = \frac{1}{\sum_{j=1}^{F} (q_j + Q_j)\theta_j} \sqrt{\left\{ \sum_{j=1}^{F} (q_j + Q_j)^2\theta_j^2 \left(\frac{s_j^2}{n_j} + \frac{S_j^2}{N_j} \right) \right\}}$$

Several Periods and Several Routes

If there are E routes, $i = 1, 2, \ldots E$, and F periods, $j = 1, 2, \ldots F$, then with an obvious notation the overall weighted mean journey time "before" is:

$$t_w = \sum_{i=1}^{E} \sum_{j=1}^{F} (q_{ij} + Q_{ij})\theta_j t_{ij} / \sum_{i=1}^{E} \sum_{j=1}^{F} (q_{ij} + Q_{ij})\theta_j$$

$\bar{T}_w$ has a similar expression and the standard error of the difference is:

$$s(\bar{T}_w - \bar{t}_w) = \frac{1}{\sum_{i=1}^{E} \sum_{j=1}^{F} (q_{ij} + Q_{ij})\theta_j} \sqrt{\left\{ \sum_{i=1}^{E} \sum_{j=1}^{F} (q_{ij} + Q_{ij})^2 \theta_j^2 \left(\frac{s_{ij}^2}{n_{ij}} + \frac{S_{ij}^2}{N_{ij}} \right) \right\}}$$

The expression for $\bar{t}_w$ can be written in a simplified form as follows:

$$\bar{t}_w = \Sigma(q + Q)\theta t / \Sigma(q + Q)\theta$$

where the symbols on the right-hand side now refer to a typical route during a typical period, and the summation is understood to extend over all routes and periods. The expression for $\bar{T}_w$ can be treated in a similar way.

Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

Dr W. H. Glanville said that he proposed to make only a few general remarks. First, he wanted to emphasize the importance of the kind of investigation into the theoretical side of road traffic which the Author had made. Some engineers were frightened when mathematics went beyond a fifth-form standard, but he could assure anyone who was willing to take the trouble that it was worth while to study the Paper in detail.

The Author had said that it was not always appreciated that in a severely practical subject such as traffic engineering, there was need for theory, and " the history of science suggests that progress in any field of research can best be achieved by a judicious mixture of practical experience, experiment, and theory." Dr Glanville considered that the Author had put that very well indeed. However, in introducing his Paper as a theoretical Paper he had been in a sense wrong, because, as Dr Glanville saw it, it was essentially a practical Paper making use of theory. What the Author had done was to advance from the pioneer investigations which had been made some years ago by Mr Adams, Mr Clayton, and others.

There was a very wide scope for investigations, theoretical and otherwise, into traffic movement, and theory could help research considerably, and therefore could help practice in many ways, for example in planning and analysing experiments, in generalizing from the experimental results, in solving problems which it would be too costly to treat experimentally, and not the least in providing an outlook which is critical of current ideas and receptive to new ones, but, of course, equally critical of them.

For instance, in connexion with the investigations conducted by the Road Research Laboratory, it was of great help to be able to investigate those matters theoretically, and it helped them to cut down the man-power required in a research. For example, from what was said on pp. 348–351 of the Paper it would be seen that a large number of runs by a test car had been required to give a reasonably accurate average journey time, even in a simple case of "before-and-after" study. On p. 351, for example, 160 runs were mentioned both before and after a particular change. If that run had taken, say, 5 minutes, with a car and a driver and two other people to make observations, it meant that perhaps 80 man-hours had been required for that simple investigation. That was a long time, and it might take an equally long or even a longer time to do the analysis afterwards, so that it was necessary to be very careful in planning the experiments, and the sort of approach shown by the Author was of great help in that respect.

Generally there was in Great Britain a growing interest in traffic problems and in the part which the provision of proper facilities for traffic played in national productivity. There was, for example, a Productivity Team which was going to America in the near future to study road problems there and how they affected the economy of the country.

For too long roads had been looked on as luxuries, which the country could ill afford. They were not, of course, anything of the kind; they were part of what was an intricate industrial machine which the nation operated for its own welfare. The machine was at present lopsided, out of balance and out of date. Anything which could be done in the way of understanding its operation better and preparing some proper basis for future development was to be welcomed.

Mr Rudd, another of Dr Glanville's colleagues at the Road Research Laboratory, had presented a Paper to the Royal Statistical Society in which it had been pointed out that the country spent about £1,200,000,000 on road transport and travel in 1950, a sum which was exceeded in 1951. That was more than 10 per cent of the national income, and about three or four times the expenditure on rail transport and travel. It was an immense business, employing about 1,500,000 people.

The surveys of London traffic made by the Road Research Laboratory some time ago had shown that a vehicle travelling through the London streets spent about one-third of the time stationary. Another interesting fact (mentioned by Mr Menzler at the meeting of the Royal Statistical Society) was that if the speed of buses in London could be increased by 1 mile per hour, the London Transport Executive would save £2,000,000 per year, and, of course, that was only a small part of the total saving which could be rendered possible by a proper system of roads.

It was clear to Dr Glanville, and he thought clear to everybody, that the best use of existing roads, as well as the proper planning of new roads, must depend closely on a full understanding of the way traffic behaved, and the Paper made an important contribution towards that understanding. In 1952, for the first time, courses in traffic engineering were being given at the Road Research Laboratory, and the Author would help with those. Dr Glanville hoped that they would be the beginning of something which would be very useful to engineers. He also hoped that the Paper would bring forth some constructive and helpful comment and criticism; it was important for the more experienced engineers to offer their full support and encouragement.

Mr W. F. Adams said that he was surprised that such a reputation as Dr Glanville described could have been acquired on the strength of one indiscretion, because he had published only one Paper on the subject. His active concern with traffic research did not go beyond 1939, since which time he had been too busy in other directions to be able to keep up an active interest in it, although the passive interest of appreciating other people's work had, he hoped, remained. He thought that the mathematics in the Paper were sound, but he would add the customary warning that, as with all mathematics, one should keep an eye on what went into the machine before deciding what to do with what came out. He would especially recommend that caution in respect of the section in the Paper on the distribution of traffic.

On p. 327 there was a proof that a traffic distribution could be random in both space and time. He welcomed that, because, after satisfying himself that it was possible to find such a distribution, but being unable to devise a proof, he had said in 1935 in a Paper entitled "Road Traffic and Probability" (which had been reproduced by the courtesy of the Road Research Laboratory), that the assumption that vehicles were distributed at random in space along a highway was different from the assumption of random distribution in time made in the Paper, but the two were not mutually contradictory. The Author had provided proof of that assertion, for which he was very grateful.

The Author had said "Moreover, the speeds of successive vehicles in space or in time form a sequence of random variables." Mr Adams would caution readers that the term "random" was there used in a sense less strict than that as he had defined it.

Referring to formula (12), which the Author had attributed to him, he would like to make it clear that he had no claim to it. The Author was only following other authority, but it was necessary to draw attention to that point, because Mr Clayton in his Paper on Traffic Calculations had mentioned the same formula without attribution, and it would be undesirable if that were to arouse unfavourable comment. In fact Mr Clayton was correct; Mr Adams had derived that particular formula, and it had come to the knowledge of one of the engineers of a signal manufacturing company, who had referred to it in one of his publications. In the meantime, Mr Clayton had shown that he had previously derived the same formula, and that it had, even before that, been worked out by American authorities. In a Paper in 1935 (unpublished), Mr Adams had said that the cycle formulae proposed by a number of authorities were all of the general form:

$$c = \frac{b}{1 - kT}.$$

That did not amount to a claim for originating the formula.

On p. 341, the Author had used a partially theoretical method employing random numbers. In 1935 Mr Adams had used numbers to investigate two problems which at that time were rather intractable. One was the proportion of periods in which a vehicle-actuated signal ran to the maximum—at that time a matter of some importance, because the linking of adjacent signals was being made to depend on that phenomenon—and the other was the estimation of delay to pedestrians crossing a carriageway. In both cases the "random numbers method" had served to dispose of tentative solutions which he had tried, but the same investigations later served to confirm more exact solutions proposed by Garwood and Martin respectively. Even at that time the use of random numbers was a known method well known to telephone-traffic engineers.

Appendix IV mentioned a condition under which the minimum cycle was not necessarily the optimum. He was very doubtful whether the condition where the minimum was not the optimum could ever apply in practice, and certainly the point was not disposed of by the example on p. 339, which seemed to him to involve assumed figures which were hopelessly unreal. In practice, very often p_1 and p_2 (in Appendix IV) were either equal or approximately so, and if they were in fact equal it was possible to prove that the optimum cycle could not differ from the minimum.

Colonel G. T. Bennett said that there was no need to apologize for a theoretical approach to road problems or problems of traffic flow and capacity of roads, because not enough in that direction had been done in the past, and road engineers would be grateful to the Author for his work and would hope that he would continue it.

In the Synopsis the Author had stated that "In considering the capacity of road systems it should be remembered that increases in the amount of traffic (the flow) generally produce corresponding decreases in speed. Capacity could be defined as 'the flow which produces the minimum acceptable average journey speed.'" Colonel Bennett thought that that paragraph was very important. It was interesting to try to understand why, when there was a larger flow, there was a lower speed. The Author might perhaps agree that the real reason was fear of accident—when there was more traffic on the road there was a natural tendency to go slower for fear of accident. That meant that if motorways were to be built in this country, for example from London to Birmingham, the safety factor would enter into the problem of design, and would offset economies. It could not be claimed as economically justifiable to build a new road from London to Birmingham merely to save accidents, because it would not be possible to save enough accidents to justify the construction of such a road; the point was that if such a road was not built to such a design that it could be used safely, road users would not use it at a sufficient speed to justify its construction on economic grounds.

Could the Author help on a problem which was directly connected with the question of the capacity of roads? About a year ago the Road Research

Laboratory, at Colonel Bennett's request, had made some investigations into the problem of accidents at road junctions, and for that purpose they had made an assumption with regard to the number of accidents which were to be expected, other things being equal, with different volumes of traffic. At his suggestion, the initial assumption had been that the number of accidents at a road junction would be proportional to the product of the two traffic streams. That was to say, if x represented the flow of traffic on one road, and y the flow on the other road, then the number of accidents would be proportional to xy. However, in a subsequent investigation, which had only recently been completed, the Road Research Laboratory had tentatively come to the conclusion that in fact accidents did not go up in that relation, but were proportional to $\sqrt{xy}$.

He did not necessarily accept that they were wholly right, but, if they were, there would be the curious result that if it did happen that at any junction $x = y$ the accidents would go up in proportion to $\sqrt{x^2}$, which would mean in proportion to x. That again would mean that the actual risk to a person using that junction would be a constant, and therefore at any junctions where the flows of traffic were equal, the risk to an individual would be the same, whether there was one vehicle or 1,000 vehicles a day going in each direction.

If that relationship between accident numbers and traffic were accepted, it would be interesting to consider what would happen where there were two junctions close together. Assuming the same total amount of traffic as in the previous case, the risk of accident at two junctions would be twice the square root of twice the risk of accident at one junction, thus: $2\sqrt{\left(\frac{xy}{2}+\frac{xy}{2}\right)} = \sqrt{2}\,\sqrt{xy}$, or about 40 per cent greater than if the entering traffic were concentrated at one junction.

That was interesting from the following point of view. Engineers sometimes had the problem of dealing with development authorities who wanted to construct houses near important roads, and it was usual to tell them that they must bring their traffic out at one point only. A developer might say "If I have two roads, with only half the traffic on each, the risk of accident will be the same, that is, half the risk of accident on each road" such a contention would appear to be correct if the risk of accident was proportional to the square of the two traffic flows, but not if the law followed the "square-root-of-the-product" rule, for then the two junctions would produce 1·41 times as many accidents as the one junction.

It was important to know whether the square-root rule did apply, because not only would it apply in the case he had mentioned, but possibly also in the analoguous case of the ordinary straight road. Did accidents vary with the volume of traffic on the road or with the square root of the volume of traffic on the road? Possibly the answer lay somewhere between the two, but it was important to know, and he hoped that more investigations would be carried out on that and similar problems, by the Author and his colleagues. Their efforts were much appreciated by highway engineers.

Colonel S. Green remarked that he had been fascinated to see some of the problems of road engineers in mathematical form. Research was of necessity slow and arduous, whilst on the other hand, engineers were beset by many " front-line " traffic problems. It was of the utmost importance, therefore, to heed the Author's plea for a judicious mixture of practical experience and theory, so as to try to get first things first. In particular, Colonel Green would emphasize the Author's statement that theory should not be divorced from experiment.

Colonel Green's own principal preoccupation lay in the heavily-trafficked centre of London, where seldom could the conditions necessary for what was called random flow be fully justified, that was, absence of nearby interruption and moderate flow. That led him to ask a question on which the Author might be able to give some information : at what distance from an interruption could the traffic be said to have out-stripped its influence and resumed a random flow ? To emphasize his meaning he referred to *Fig. 4*, which showed traffic leaving an intersection by parallel lines. The fact could be accepted that, as the Author had said, sooner or later something would happen in time and space to those parallel lines.

That aspect of the question was very important when considering interconnexion between signals, and attempting to improve time gaps for pedestrians in densely built-up areas where there were frequent intersections, a speed limit of 30 miles per hour, slow-moving traffic regulations, and a hard core of public service vehicles which by virtue of their business had fairly constant journey times. At the commencement of a green period at traffic lights in a heavily-trafficked built-up area, the first vehicle should not move at more than 30 miles per hour, and the last vehicle might be a straggler going about 10 miles per hour. Within that limited range the main block of traffic travelled, and it seemed obvious that if nearby installations were co-ordinated, there would be intermediate points with adequate time gaps apart from stragglers. It was that aspect of flow in built-up areas in which he was very interested, and he could also see the possibility of improving time gaps for pedestrians by remote connexion.

If *Fig. 2* had been prepared on the same basis as *Fig. 1* it would probably be found under journey speed that the mean would be of the order of 11 miles per hour, and that deviations from the mean, —2 and +2, would probably give journey speeds of, say, 5 and 16½ miles per hour. Would the Author say whether that was correct, because it emphasized the point which Colonel Green was trying to make.

On p. 336 a formula had been given about which he was a little doubtful. He had tried that formula on one or two roads of which he had knowledge, and it did not appear to fit. Would the Author offer some advice, and in particular, would he try to apply it to the Victoria Embankment?

Fig. 7 emphasized the delays as between a 30-foot- carriageway and a 48-foot carriageway. Those facts were probably well known, but it was nevertheless a very interesting diagram, and there was scope there for experiment by engineers. A good example of the merging of lanes of traffic was at Hyde Park Corner, where the eastbound traffic lined up four lanes abreast, and when released into Piccadilly soon merged into one lane In roads capable of taking less than four lanes (including 50-foot roads) he would like to see more experiments with refuges at intersections out of centre, to allow for two lanes to enter the intersection in each direction, provided adequate merging length could be arranged. The effect of that artifice, if successful, would be to close the gap between the two graphs in *Fig. 7*.

On p. 353 the Author had dealt with one-way schemes and compared journey times. Another point to be taken into consideration was the possibility that a one-way scheme might give increased capacity. He thought he was right in saying that Dr Smeed, in reporting on a case in Slough, had said that although the scheme had resulted in extra traffic times, it had, in one direction at least, encouraged a very high percentage of additional traffic to use it. It would seem therefore, that additional capacity might be obtained by such a scheme as a result of which more vehicles were attracted to a longer distance.

Finally, Colonel Green referred to the Author's remarks about future work, and in particular the linking of signal installations. It was necessary to-day to deal with heavy pedestrian movements across heavy traffic flows, and time gaps for pedestrians were a very important consideration. If those time gaps could not be improved by remote linking—and that brought in the question which he had raised earlier, of the distance at which randomness occurred—special time gaps would be required in the interests of safety sooner or later, and the frequency of those time gaps would inevitably determine the safe speed for vehicles on any given street. In other words, as and when all those time gaps were provided, every road would by virtue of its character, have its own self-imposed speed limit.

Mr D. A. de C. Bellamy said that the outstanding importance of the Paper was the collection of many theories in one place, showing that there was very much more in traffic work than the mere application of rule-of-thumb methods. The Author was to be congratulated on his very lucid exposition of the difference between the space-mean speed and the time-mean speed.

There were three questions which were always propounded by those making a first approach to traffic problems, (1) what was the capacity of a road, (2) what was the capacity of an intersection, and (3) what was the capacity of a roundabout; but there was no unique answer—it depended on many factors. Those capacities, fortunately, were not of very much value in practical work, because it was very much easier to get at what was wanted by a somewhat different approach.

The capacity of a road in a town depended partly on the volume of the cross-traffic at intersections and partly on the speed at which traffic could move at saturation, which in turn depended on the spacing of the junctions, the volume of traffic, the volume of pedestrian traffic crossing the road, the number of effective lanes and the distribution of standing vehicles.

No details had been given in the Paper of what measurements had been made, or where the observations had been taken, in arriving at the results given in *Fig. 3*, so that it was difficult, and perhaps rash, to make any comments. The formula at the top of p. 336 he found interesting. In the denominator the effective width of the road was given by the deduction of one waiting lane. Were waiting restriction orders in operation in the streets studied, or was there some other explanation? Another point was that all the observations stopped when the traffic was running at one-seventh saturation. Engineers were familiar with extrapolation and aware of its dangers, but it was something which they had been using for a long time, and without employing it he doubted whether the big structures now in existence, such as the Sydney Harbour Bridge, could have been developed. It was curious to note, however, that if each of the sloping straight lines was continued to the x axis—the earlier ones had not very far to go—the maximum flow of traffic was obtained when the speed was zero. In other words to pass a large volume of traffic it must be stopped. That was contrary to the experience of Mr Adams and Mr Purkiss, of Marylebone, 15 years ago, because they had raised the speed of traffic from 4 miles per hour to 6 or 8 miles per hour and had got more traffic through. It was unlikely that the lines would bend suddenly to the right, but they might bend to the left and pass through the origin, giving no flow for no speed, which was at any rate a rational result. They looked as though they ought to be straight lines, but had any check been made to see whether a straight line was a proper one to fit? It might be that some sort of parabola would fit the points better, but he would not waste the Author's time by asking him to fit a parabola because in view of the number of variables involved the curve was going to be somewhat complicated.

On the open road the driver's convenience was the controlling factor, and not the maximum volume which could be passed. The layout provided was dependent on what the client was prepared to pay for, and all that the engineer could do was to advise his client whether the conditions would be good, indifferent, or bad, and how far they were likely to depart from the ideal.

Dr R. J. Smeed said that he wished to emphasize the importance of theoretical analysis. In time such work always became the stock-in-trade of the practising engineer. Working out the theoretical problems involved was, he believed, one of the ways in which the Road Research Laboratory could be specially useful. Every highway engineer concerned with traffic problems could and should make a number of investigations, but few of those engineers had the time or the inclination to consider the basic theoretical work involved.

It was theoretical analysis which emphasized the importance of the difference between the two ways of calculating mean speed mentioned in the Paper. One of those ways was to calculate the ordinary arithmetic mean of the journey speeds. The other corresponded to taking the arithmetic mean of the times taken to traverse the distance under consideration and to use that time as a denominator to calculate a mean speed over the distance. He had made up an example to show the very large difference in mean speeds which could be obtained by using the two methods. Every schoolboy was taught that such differences existed, but few people realized how great a difference could occur in practical cases.

The problem which he had made up, was that during a certain period nine vehicles travelled at 30 miles per hour and one at 2 miles per hour, over a road $\frac{1}{2}$ mile in length. The problem was to find the average journey speed. In the one method it was obviously 9×30 miles per hour plus 1×2 miles per hour, divided by $9 + 1$, or $\frac{9 \times 30 + 1 \times 2}{9 + 1}$, which worked out at 27·2 miles per hour. By the second method, the journey time of vehicles at 30 miles per hour was 1 minute, and the journey time of the vehicle travelling at 2 miles per hour was 15 minutes, so that the average journey time was $\frac{9 \times 1 + 1 \times 15}{9 + 1}$, which worked out at 2·4 minutes. An average time of 2·4 minutes for traversing $\frac{1}{2}$ mile meant a speed of 12·5 miles per hour. The difference was enormous. The Author had mentioned that the difference between mean speeds was usually between 6 and 12 per cent, but the example just given showed that it could be very much greater.

At the Road Research Laboratory, a number of theoretical problems in addition to those which the Author had mentioned were under consideration. A Paper * had recently been published, for example, giving formulae for the average delay and distribution of delays when pedestrians wished to cross the road but required a given gap in the traffic. The mathematical solution of the problem of two opposing streams of traffic wishing to pass along the same stretch of road, a problem which occurred when there was a single stream working on a limited stretch of road, had also been found. It was unnecessary to go into all the possible applications of that work, but it was clearly of great value in deciding the answers to such a problem as when a pedestrian traffic light was justified instead of an uncontrolled crossing. However, the formulae obtained showed clearly that the mathematical solution of such problems was very complicated in most cases, and

---

* J. C. Tanner, "The delay to pedestrians crossing a road." Biometrika, vol. 38 p. 383.

that the ultimate solution of such problems would have to be found by other methods. Development had therefore been started of a machine which would calculate the solution of many traffic problems, and progress had been made. In particular, it was hoped that it would solve the problem of the optimum cycle of traffic lights.

Into that machine could be fed data for the times of arrival and rate of discharge of the traffic from any particular stream, and the period during which any stream could discharge; the machine would then calculate the delay. It was hoped in that way to find the optimum phasing and cycle time at a number of traffic lights. It might well be that the methods at present in use to determine those quantities were as efficient as could be desired, but it would be useful to check that point.

Dr Smeed referred to the point which Colonel Bennett had raised about accidents at 3-way junctions, in case the Author was not fully *au fait* with that work, since he was not working directly on it. It was true, as Colonel Bennett had said, that the Road Research Laboratory had found that for certain 3-way junctions the accidents seemed to vary as the square root of the product of the flows instead of, as might be expected, as the product of the flows, but it could only be said that that seemed to be true for the range of flows considered. It would certainly not be correct to extrapolate the result either to junctions with very few turning movements or to junctions with vast numbers of them. Similar laws had been found in a number of other cases, which did not operate in quite the way which might be expected from first principles. For instance, it had been found some time ago that the accident rate per vehicle-mile in built-up areas did not seem to depend very markedly on the amount of other traffic, as from first principles it might have been expected to do.

Mr A. J. H. Clayton agreed with those who had said that there was no need to apologize for dealing with the theoretical aspect of traffic. He would particularly endorse what Dr Smeed had said on that point, and would go a little further and say that, if anything, the work of the Road Research Laboratory was not theoretical enough; there was a tendency in some research work, no doubt of necessity, to take observations, generally where there were large numbers of variables, to plot them and so get a result. *Figs 3* and *7* of the Paper were very good examples.

What the engineer wanted was a working theory; he very often had to use unchecked theories in order to obtain results before the experimental work could be done, and even to extrapolate from the experimental data when it became available.

Experience proved advantageous, particularly in research, in providing knowledge of the probable variations in value of various items to be investigated and the most useful limits to place on the results. As an example, Mr Clayton referred to the question of using the assumption of the random distribution of traffic or the assumption of uniform distribution. Both those assumptions, like the assumption that plane surfaces remained plane after bending, were not generally true, but they were good working assumptions, and in practice the one should be taken which gave the result nearest to correct, which result would often be much nearer the truth than the assumption itself was to the truth.

In *Fig. 6* the Author had shown some very useful work on testing actual random flow against the cycle of operation of signals. Mr Clayton, like him, had noticed that the actual delay observed at intersections was rather greater than that found by Mr Clayton's own formula, but he did not think that that was the result of assuming uniform flow where it was in fact random. He had not regarded flow as uniform, but merely as being near enough to uniform in its arrival at the intersection. He had only been considering, and he thought that it was only necessary to consider, the conditions of fairly dense flow in working out what happened at fixed-time traffic signals. *Fig. 6* should stop short at the 50 per cent point. An outgoing flow of more than 50 per cent of the incoming, or at intervals of more than 3 seconds, was not likely to occur with nine hundred vehicles per hour coming in; it was not practical. That was where practical experience helped in deciding where to stop the curves. He thought that the fact that traffic delays at intersections were greater than would be suggested by simple theory was due to extraneous factors such as right-hand turns, pedestrians crossing the road at the wrong time, and so on.

In dealing with the awkward mixture of varying speeds, varying intervals, etc., which occurred in a traffic stream, was the mean speed, however determined, always the right figure to use? The results obviously had to be reduced to something intelligible, something which could be understood by the layman to whom the answers were to be presented, and it was necessary to consider what was the right figure to use. By and large, the benefit to the community was probably best measured by what happened to the average driver, but it was not always so, particularly in cases of "before-and-after" studies of re-arrangements of traffic or of road improvements.

He would give as an example the Vauxhall cross intersection. Those who remembered it before the improvement would recall that in those days it was a bad place, and in the morning rush-hour traffic was liable to be held up for 10 or 15 minutes, whilst sometimes there was very little delay. Nevertheless, drivers and passengers had to allow 15 minutes if they were to be on time at their destinations. If the Road Research Laboratory Traffic Branch had been in existence at that time, careful observations might have been taken in order to determine whether the roundabout, which in spite of its awkward shape seemed to be working very well, was in fact any better. It might be found to take about 1½ minute to go round that roundabout, but the average delay time before it was instituted might not have averaged more than 1½ minutes. Could that be regarded as any improvement? In such cases what should be taken was not the mean delay, but the 90 per cent delay; in other words, the maximum delay that 90 per cent of the traffic was likely to experience leaving aside a few very unlucky ones. Similarly, a better description of the conditions set out in *Fig. 1* would be that 80 per cent of the vehicles went at speeds between 20 and 50 miles per hour, than a statement of the mean speed.

Mr J. T. Duff expressed strong agreement with the Author on the value of the theoretical approach to traffic problems. It had been said, he observed, that anyone could build a bridge which would be strong enough, but only an engineer could build one which would be just strong enough. A similar remark would apply to traffic engineering; whilst rule-of-thumb methods would give results, the greatest economy and efficiency could be achieved only when proper attention was paid to the theory.

As traffic increased, it became more than ever necessary to ensure the maximum efficiency in all phases of traffic engineering. Theoretical work with regard to traffic could be broadly classed under two headings. The first was the application of standard statistical techniques such as the Author had illustrated in the section of his Paper dealing with " before-and-after " studies. Traffic movement was characterized by the large variances which occurred, and, to avoid the uneconomically large sample sizes which it had been necessary to use in the past, more use must now be made of efficient statistical techniques. Further work on the theoretical side lay in adapting standard methods or devising new ones to deal with fresh problems. On the practical side, much work required to be done in presenting such techniques to the practising engineer in a more digestible form.

The second broad heading was the creation of mathematical models of traffic behaviour, of which the Author had given several examples in the Paper. A large number of interacting factors would be found in even the simplest problem, and made that an exceedingly difficult task; either the model was too simple and failed to give a convincing description of traffic conditions or it was too complex to be given an adequate mathematical treatment. Nevertheless, some useful progress had already been made, and the results had been made available to the practising engineer. The theory given in the Paper had in the main been built up over the past 20 years but, outside a handful of specialists, had not been widely appreciated in Great Britain. If the Paper helped to make the engineer more familiar with existing knowledge it would have served a very useful purpose, but he was afraid that the symbols used would discourage many engineers.

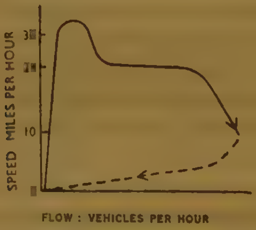
In the section headed " Capacity of Road Systems " the Author had said " However, these conditions are extremely artificial and take little account of what drivers actually do." Mr Duff thought that that remark might well be transferred to the beginning of the section headed " Frequency of Overtaking." For example, it seemed to him unlikely that a driver travelling at 41 miles per hour would desire to overtake a vehicle travelling 40 miles per hour. He would probably be content to continue driving behind it, but, if he did decide to overtake it, it would not be at

41 miles per hour—he would accelerate to a greater speed. With a continuous distribution, the problem was even more difficult.

Turning again to the section on " Capacity of Road Systems," Mr Duff failed to see any difficulty in the idea that at zero speed the flow was zero. Apart from being obvious physically, any engineer who had studied the curve for saturation capacity would be familiar with the idea that, as the speed increased from zero, the saturation flow increased from zero to the maximum of 15 to 20 miles per hour, and thereafter fell again as the speed increased further.

The formula on p. 336, to which several speakers had referred, Mr Duff found a little difficult fully to appreciate. He did not understand, for example, the sudden discontinuity at 24 miles per hour. Presumably the formula applied only to London streets, where the maximum speed was governed by the presence of numerous intersections, concentrations of pedestrians, and other factors, but those factors must also be operative at the lower speeds, and should be reflected somewhere in the formula. Again, he did not follow why there was the discontinuity at 10 miles per hour, unless it was that no speeds lower than that had actually been measured. It seemed to him that the curve would be more of the type shown in *Fig. 9*. When there were no vehicles the speed and flow would be zero, so that it would be necessary to start at the origin. When there was one vehicle it could travel at any speed it liked, but presumably with a limit of 30 miles per hour, since that speed limit operated in most London streets. The curve, therefore, must rise very steeply up to 30 miles per hour. As soon as there was any concentration of vehicles it would come down to, say, 24 miles per hour, where it would remain steady for a time, and then start to fall again, as the Author had shown, down to 10 miles per hour. What happened after that was a matter of conjecture, but it seemed to him that the curve might follow the dotted line shown in *Fig. 9*, so that as an increasing number of vehicles wanted to enter the section, a point was reached where the flow was reduced to zero and there was a complete jam. Even then, however, the formula was still unconvincing, because on that theory all that it was necessary to do was to introduce a speed limit of about 5

Fig. 9



miles per hour, and then most of London's traffic problems would be solved.

In the section of the Paper on "Signal-Controlled Intersections," he could not see that much had been gained by changing the familiar symbols. The engineer who was already accustomed to using the saturation capacity of, say, 1,500 or 1,800 vehicles per hour in calculating his cycle of times would be puzzled to find that he should use a different figure for each approach road, and indeed for every problem. In practice he would multiply his 1,800 by the number of lanes (which might be fractional), which would in fact be what he had always done.

The footnote on p. 350 stated that $\frac{d-\delta}{s(d)}$ had a "t-distribution." In fact the expression for $s(d)$ for small sample work differed slightly from that given on p. 349.

The Chairman pointed out that it would be unwise to ignore the extent to which the human element entered into the problem. He hoped that it would be possible to produce formulae to deal with most of the factors, but even with the best formulae there would still remain the questions of the make of the car and the way the driver handled the car, which would have to be taken into consideration.

The Author, in reply, regretted any misunderstanding which he might have caused by the use of Mr Adams' name in formula (12). Referring to the question of the reality or unreality of the figures used in the example on p. 339, in which the optimum cycle was shown to differ from the minimum cycle, he disagreed with Mr Adams. The figures quoted corresponded to a saturation flow of 2,500 vehicles per hour of green time in each direction on a major road, and 1,000 vehicles per hour on a minor road. Those figures would not be unreasonable at the intersection of a 50-foot road with a 20-foot road. The actual flows assumed were 500 vehicles per hour in each direction on the major road and 100 vehicles per hour on the minor road. There were certainly a number of intersections of that type on main arterial roads.

Colonel Bennett had suggested that a higher traffic flow produced a lower speed, because drivers slowed down for fear of accidents. That was undoubtedly a factor, but in heavy traffic it was physically impossible for most vehicles to make all the overtakings necessary to maintain their desired speeds. Colonel Bennett had also raised the question, to which Dr Smeed had referred, of the frequency of accidents at three-way junctions varying with the square root of the flow-product, rather than the product itself; Dr Smeed had mentioned that the accident rate per vehicle-mile in built-up areas did not vary markedly with the amount of traffic, which it might be expected to do from first principles. Both those results suggested that the accident risk at each encounter with another vehicle was reduced as the number of encounters increased, in such a way that the risk incurred in travelling one mile, or passing through one intersection, was kept constant. However, the result was a purely empirical one, and the correct interpretation could only be found by further investigation.

Colonel Green had asked how long it would take traffic to reach a random distribution after leaving a signal. The Author hoped to investigate that problem in detail, but some rough results could be given for a typical case. If a heavy flow of vehicles left a signal with a 30-second green period in a 60-second cycle and had an average speed of 15 miles per hour with a 20-per-cent coefficient of variation, then the distribution would be quite random after a distance of 1 mile. At a distance of ¼ mile, however, the traffic distribution would show marked peaks and troughs.

Mr Clayton had doubted whether a random flow was a reasonable approximation in heavy traffic. The figures just mentioned suggested that intersections at distances of ¼ mile or less would not receive either a random or a uniform flow. However, the effect of turning traffic, both at the controlled intersections and at others, would be to fill up the troughs. As Mr Clayton had said, traffic was usually somewhere between uniform and random; but in many cases an assumption of randomness was nearer the truth than an assumption of uniformity. Comparison of calculated with actual delays had shown quite good agreement between random results and observations, whereas the uniform results did not agree at high flow-levels.

Colonel Green had made an interesting suggestion for dividing a 3-lane road at an intersection into two lanes approaching the intersection and one leaving it. It would be useful to try the experiment at several places and see how well it worked. A good deal would depend on the amount of turning traffic; if there was little, there might be congestion at the exit from the intersection.

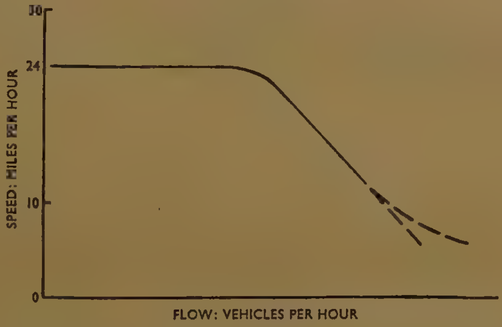
Colonel Green had asked what the variation in speed shown in *Figs 2* would be in terms of miles per hour. The answer depended on the distance travelled. Taking the average of 11 miles per hour suggested by Colonel Green, on one mile the variation of +2 standard deviations from the mean would give a range of 6 to 16 miles per hour; on ½ mile it would be 4½ to 17½ miles per hour.

It had been suggested that a one-way scheme might increase capacity although it also increased the journey time at the present-day flow. Assuming capacity to mean the maximum flow which did not produce an intolerably long journey time, it was quite possible for that to happen. However, as a general rule increases in journey time seemed to be accompanied by decreases in capacity.

Fig. 3 seemed to have caused some difficulty. It should be emphasized that it presented the results of observations, and that the formula on the top of p. 336 was an empirical one. The fact that a distance of 6 feet was subtracted from the road width in the formula did not mean that a single waiting lane had been assumed. A number of interacting factors were involved, and it was unwise to identify a figure in the formula with a particular factor. Mr Bellamy and Mr Duff had speculated about the behaviour of the lines in *Fig. 3* outside the range of flows studied. The Author did not think that the peak speed of 30 miles per hour at very low flow shown in Mr Duff's diagram (*Fig. 9*) actually occurred. Observations had been made at flows of less than fifty vehicles per hour, when interference between vehicles should be negligible. The explanation was probably that not all drivers wished to travel at 30 miles per hour nor were all vehicles able or permitted to do so; bends in the road, pedestrians, and other hazards would also reduce speed.

The broken line in *Fig. 9* could not have been obtained owing to the method of obtaining the results. The average speed was found for each level of flow; individual vehicles might travel faster or slower, but there could only be a single average speed. The flow had been regarded as the independent variable and the speed as the dependent variable. The Author thought that the curve might follow either of the paths shown in *Fig. 10*, but in such a case extrapolation might be misleading. There was no suggestion that flow could be increased by controlling speed at a reduced level. The point was that by tolerating reduced speeds, higher flows could be obtained.

Fig. 10



Mr Clayton had been misled by an error in *Fig. 6* in the advance copy which the Author regretted. The correct figure for the flow was now shown by a corrigendum inserted at the side of the Figure. It should perhaps be pointed out that term " maximum flow " meant the maximum which could enter the intersection without the formation of a steadily increasing queue. With the revised figure for inflow the diagram need not be stopped at the 50-per-cent point.

The Author agreed with Mr Clayton that in some cases the journey time which was exceeded on 10 per cent of occasions would be a better criterion in a before-and-after study than the average journey time. That would be so if the emphasis was on planning a journey in order to arrive at a given time, if a risk of 1 in 10 of being late was acceptable. However, in making calculations on the economic benefit to be obtained from an improvement, the Road Research Laboratory were hoping to evaluate time saved in terms of money, and for that purpose the average time saved was usually required.

In reply to Mr. Duff, the Author agreed that with reference to the footnote on p. 350, the formula for $s(t)$ was slightly different from that required to give the t distribution.

He did not think that a driver travelling at 41 miles per hour who came up behind another travelling at 40 miles per hour was often content to stay behind him. The overtaking speed would, of course, be higher than 41 miles per hour, but that would introduce a negligible error in the formula for the number of overtakings.

Correspondence on the foregoing Paper is now closed and no further contributions, other than those already received at the Institution, may be accepted.—Sec. I.C.E.

RAILWAY ENGINEERING DIVISION MEETING

29 January, 1952

Mr J. C. L. TRAIN, M.C., M.I.C.E., Chairman of the Division, in the Chair

A discussion on the subject of

"Continental Railway Civil Engineering Practice"

was initiated by the Introductory Notes and, upon the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Introductory Notes :

(1) "**Observations on a Visit to the Netherlands Railways in 1950**"

by

JAMES GORDON FARQUHAR INGLIS, M.A., A.M.I.C.E.

(2) "**Observations on a Visit to the French Railways in 1949**"

by

ROBERT GEOFFREY THURTLE, M.B.E., B.Sc.(Eng.), A.M.I.C.E.

Introductory Note No. 1

"Observations on a Visit to the Netherlands Railways in 1950"

by

JAMES GORDON FARQUHAR INGLIS

SYNOPSIS

As part of the interchange-of-staff scheme of the British and Netherlands Railways, the Author visited Holland for four weeks during 1950. The note is merely a record of some of the Author's observations at that time, and does not purport to be a concise description of the Netherlands Railways' current organization and practice.

The first part of the Note deals with the organization of the Civil Engineer's Department of the Netherlands Railways, with a brief explanation of the system of education of the staff.

The second part deals with some technical aspects of Dutch practice. The Author has confined himself to the subjects of soil mechanics, track stability, and some civil engineering aspects of electrification, all of which are of recent evolution and are likely to be more extensively developed in the future.

INTRODUCTION

THE railways in the Netherlands were brought under state ownership over 30 years ago when the two large companies, then in control of the railway networks, were merged. All development since then has been on a national basis and, although some rearrangement took place after the recent war, the general organization has become quite stable.

The Head Office organization of the Netherlands Railways is now concentrated at Utrecht, with district offices maintained by the various departments in eight main railway centres.

EDUCATION

Education in the Netherlands is under state supervision and largely state operated, although there exists a great number of denominational schools.

There is considerable similarity between the systems of primary and secondary schools in Britain and the Netherlands. The artisans, however, do not become apprenticed when their ordinary education is finished, but go to special Trade Schools where they take a four-year course, of which only 1 year is spent away from the school as apprentices.

For the lower grades of technical staff there are "Middle Technical Schools" which provide an extension to the secondary school course, specializing in, for example, engineering theory and practice. The course at these schools also lasts four years, and again 1 year is spent away from the school obtaining practical experience.

Those students going to a University take the longer secondary school course. Engineers subsequently go to Delft, the only engineering University in the Netherlands, where they take a five-year course in one particular branch of engineering before graduating. The Author formed the opinion that the civil engineering course at Delft is very comprehensive and detailed. It includes such subjects as drawing office training and practice, practical design in many fields including, if selected, railway engineering in some detail, but very little background in either mechanical or electrical engineering.

Academic education continues for rather longer in the Netherlands than in Great Britain, especially in the higher grades. This education, however, includes practical work which would normally be carried out after leaving school or a University in Great Britain.

CIVIL ENGINEER'S DEPARTMENT

General

One very important difference in the grades of staff, arising from the different educational systems of Great Britain and Holland, must be pointed out.

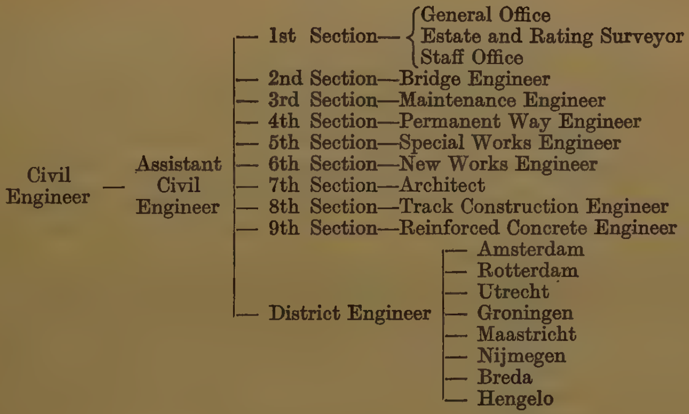
In the Netherlands Railways, the technical staff of the Civil Engineer's

Department is divided into three classes instead of the two in British Railways. Whereas in Great Britain a man joining as a platelayer can become a chief inspector, in Holland he can normally reach only the grade of ganger. The second class in Britain comprises staff from draughtsman to Civil Engineer, whereas in the Netherlands a draughtsman or inspector can become only a chief draughtsman or chief inspector, because it is necessary to be an engineering graduate in order to belong to the top grade, of which the starting point is junior engineer.

Organization

The organization of the Civil Engineer's Department (*Fig. 1*) is very similar to that in British Railways, although there are more head office sections. The first section takes the responsibility for setting out large works as well as making surveys. The fifth section was formed after the war to undertake the supervision and control of large contracts for the repair of war damage, in order to avoid temporary expansion of other departments. Its function was primarily that of consulting engineers and by 1950 it had become separated from the Railway, although some of its officers were railwaymen on loan.

Fig. 1


Civil Engineer
Assistant Civil Engineer
1st Section
General Office
Estate and Rating Surveyor
Staff Office
2nd Section—Bridge Engineer
3rd Section—Maintenance Engineer
4th Section—Permanent Way Engineer
5th Section—Special Works Engineer
6th Section—New Works Engineer
7th Section—Architect
8th Section—Track Construction Engineer
9th Section—Reinforced Concrete Engineer
District Engineer
Amsterdam
Rotterdam
Utrecht
Groningen
Maastricht
Nijmegen
Breda
Hengelo

ORGANIZATION OF CIVIL ENGINEER'S DEPARTMENT

One important difference from British practice is that all Heads of Sections and District Engineers are considered to be equal, and the districts, which correspond exactly with Traffic and Mechanical and Electrical Engineer's districts, have been adjusted to be of approximately equal importance.

Another interesting feature of Dutch practice is that all officers of the Civil Engineer's Department, that is the graduate engineers, are divided into grades. Heads of Sections and District Engineers are designated

"Chief Engineer." Their Assistants in head office sections are always ranked as "Head of Section," whilst in districts this is less frequently the case. The subordinate grades are "Engineer first class," "Engineer second class," and "Junior Engineer." Promotion is automatic up to the rank of Engineer first class, and is largely dependent on length of service.

To give an idea of the size of the organization, there are about sixty graduate engineers in the Department, and including the Civil Engineer and Assistant Civil Engineer, eighteen or nineteen of these are in the top grade.

Lower Technical Staff

The lower technical staff is drawn from the Middle Technical Schools, and comprises the inspectors and draughtsmen. These two grades are regarded in the Netherlands as being equal and interchangeable, at least for the younger staff. As the men get older they tend to specialize with a bias towards either office or outside work. It will be appreciated that Dutch inspectors are of a very different type from those in Britain, although there is not so much difference between the draughtsmen.

The inspectors are comparatively younger, joining at about the age of 22, and are, in general, much more skilled technically, being able to prepare drawings, use surveying instruments, and work out simple design calculations, and are therefore able to deal with the smaller problems on their own initiative. They lack, of course, the practical experience of their British counterparts and are less used to the supervision of labour. They are eligible for promotion only to chief inspector or chief draughtsman, and are never able to reach the rank of engineer.

There is not the same distinction between works and permanent way inspectors (probably because of the difference in educational systems referred to previously), since in Great Britain permanent way inspectors are promoted from platelayers, and building inspectors from artisans, whereas in Holland both start as junior inspectors. Among the senior inspectors there is, however, more specialization, particularly in bridge work.

Contracting Arrangements

Owing to the smaller size of the Netherlands, head office sections in Utrecht undertake much of the work which is done by the district offices in Britain. Also, very little work, apart from routine maintenance of the track, is done by direct labour, the major part being carried out under contract. There is also a system in the Netherlands almost without parallel in Brtiain, although one region of British Railways has developed a somewhat similar organization.

This organization is in the nature of an independent contracting firm controlled by the Netherlands Railways, but connected only loosely with the Railway organization, and whose officers are not Railway Officers.

It tenders for railway works in open competition, and is also employed for rush jobs and certain intricate works, especially in cases where it is not possible to obtain a reasonable tender from an outside firm of contractors.

It is also called upon to supply extra temporary labour when required. This last function is important, since in Holland the permanent railwayman enjoys a somewhat special position. He is regarded as an essential worker and strike action is prohibited. In return, his conditions of service are carefully supervised and controlled by the appropriate Government Department. It is thus extremely difficult to vary the number of railway employees according to the work in hand, whereas this independent firm can obtain labour in the usual way, and can therefore alter the size of its labour force according to circumstances.

SOIL MECHANICS

Soil Mechanics in Holland

The science of soil mechanics, although of recent development, is of great importance in the Netherlands owing to the generally poor bearing quality of the ground. A national central laboratory and soil mechanics service has been formed at Delft on the same lines as the great Dutch hydraulic research service, which compares with the British National Physical Laboratory at Teddington.

Organization for Soil Mechanics in the Railway

The Netherlands Railways have been quick to evolve a soil mechanics practice suitable to its particular needs. One of the sections of the Civil Engineer's Department is called the Track Construction Section (see *Fig. 1*), and undertakes the construction, "heavy maintenance," and repair, of earthworks and of track formation. It is also responsible, at the present time, for the civil engineering aspects of electrification extensions. Much soil mechanics work, therefore, comes naturally within its scope, and to avoid duplication of effort, resources, and skilled personnel, it makes investigations for, and gives advice and assistance to other sections on soil mechanics matters.

Thus soil mechanics has become an important integral part of the work of one executive section and not the specialized sideline of a larger section as is generally the case in Great Britain.

Investigational Methods

The two chief soil mechanics problems confronting the Netherlands Railways are the provision and maintenance of a stable track-formation and the design of adequate foundations for buildings and bridges, these foundations usually involving the construction of a raft or the provision of piles.

Two standardized field tests are used by the Railway :

(1) Cone testing; in which a cone of standard diameter and angle is forced down, and the resistance to pressure measured at intervals.

(2) Sampling; either with a simple screw auger of small diameter with a raised lip to retain the material on the thread, or with a special small-diameter sampling tube which, by a simple mechanism, can be made to open to receive material at any desired depth.

These tests are supplemented where necessary by:

(*a*) consolidation tests; and

(*b*) "cell" tests, a development of the tri-axial tests;

although these last two are carried out by the National Laboratory, Delft, and not by the Railway.

Investigational Procedure

Site investigations using tests (1) and (2) are carried out by specially trained inspectors of the Track Construction Section using unskilled labour obtained locally, the necessary equipment being sent out from the Utrecht headquarters as required.

After examination, all samples collected at the site are put into ordinary containers, not specially airtight, and sent back to Utrecht where they are put into a storeroom to dry out. The inspector who has carried out the field-work passes over his results to a draughtsman who makes any necessary calculations and checks the work already done. Part of this draughtsman's duty is to examine the samples when they have dried out and check the descriptions given to them by the inspector in the field.

So far as possible, information resulting from these investigations is produced in standardized form, calculations and descriptions both being written up on printed sheets. This is intended to simplify the work, eliminate chance errors, and assist in the keeping of records.

Recording of Data

The keeping of records is regarded as being an important duty of this section and one which is of great use to other sections when they have foundation problems.

By having available, in standardized form, all investigational data, classified according to routes, and by referring to the Geological Survey maps, it is possible to predict fairly accurately the nature of the ground to be found at any point on the system. This is of especial value when there is neither time nor justification for a separate investigation.

TRACK CONSTRUCTION AND STABILIZATION

Of the two principal types of problem dealt with by the soil mechanics section, track construction and stabilization works are probably more interesting than foundations because of their necessarily individual treatment. Three of these works visited during execution in 1950 will therefore be briefly described.

Rotterdam—Nieuwerkerk

The work on this site involves the construction of 6½ miles of new double-line railway, mainly over polder land 15 feet below sea-level. The objects are to improve traffic facilities by the concentration of lines in the Rotterdam railway network; to enable Rotterdam Maas Station to be closed; to by-pass an existing line having a weak formation and, incidentally, to eliminate a number of level-crossings.

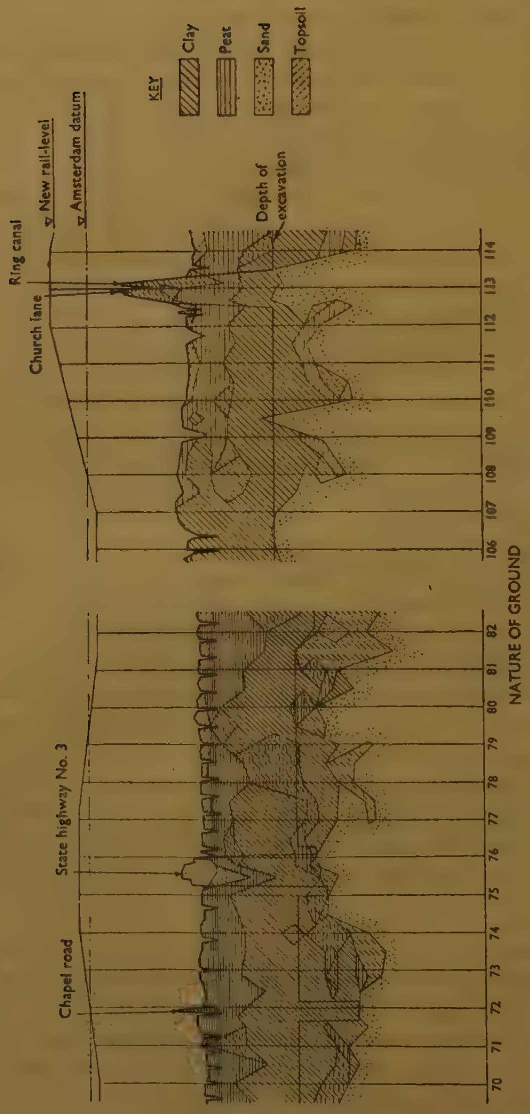
A preliminary survey of the site was carried out using the cone-testing apparatus and the sampling apparatus. As a result, it was decided to excavate to a depth of as much as 15 feet in some places and to replace the weak material by sand. A section of the diagram showing the results of this survey is reproduced in *Figs 2* and the close agreement between the type of material found and its resistance to penetration may be noted.

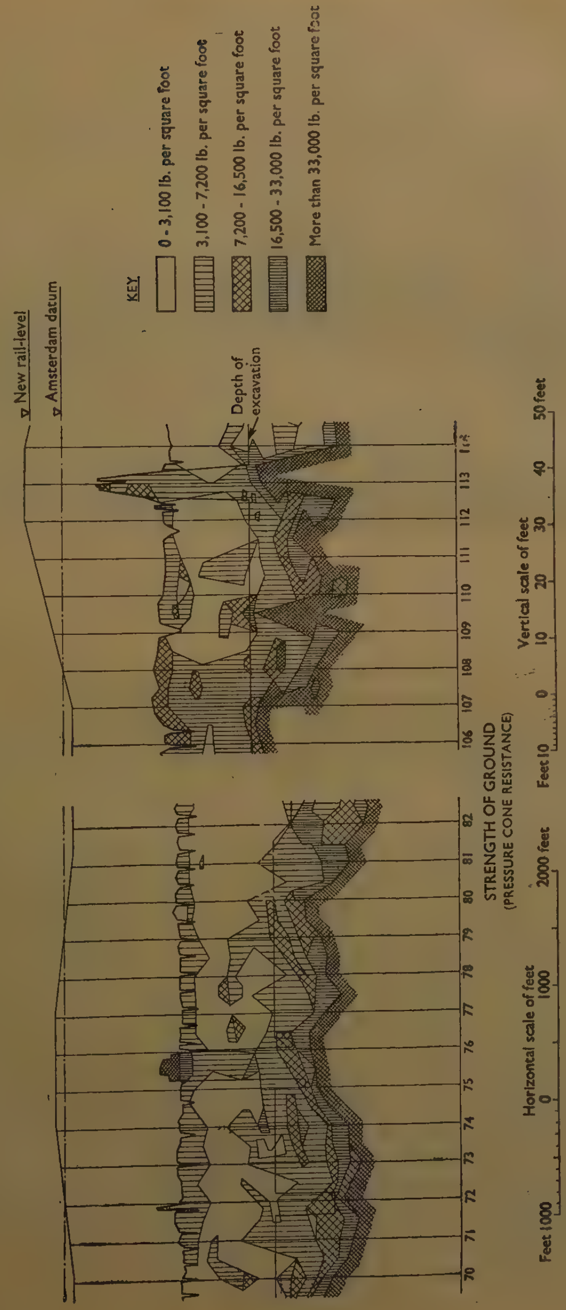
The method of construction adopted for this work was very different from normal British practice. A "pond" was first formed by dragline excavators in the polder, the standing-water level in which is normally kept below ground level only by continual pumping out into the ring canal which surrounds it. A "fleet" of dredgers, barges, and tugs was hauled down into this pond from the canal. Bucket-dredgers then advanced through the polder, along the line of the new railway, discharging the excavated material into barges, from which it was pumped by a suction-dredger through pipe-lines to raised depots, in which the deposited clay and peat was so disposed as to ensure subsequent fertility. The "canal" or trench thus formed by the bucket-dredgers was filled to ground-level with sand supplied from the River Lek, some of the sand being pumped from points up to 5 miles away. Since this sand was completely saturated when first deposited, it was necessary to allow it to drain before using it in the final embankment in order to avoid overstressing the ground alongside. It was therefore pumped to depots spaced at intervals along the new line, causing inevitably a certain amount of local heaving of the surrounding ground, from which it was distributed, after being drained, by narrow-gauge hopper-wagons to form the final continuous embankment.

Gouda—Oudewater

The purpose of the works between these stations was to strengthen the formation of a double-track main line carried on an embankment over peaty subsoil. Several types of blanketing and stabilizing methods had previously been used on other sections of this route, but in 1950 the method in use was the construction of a continuous reinforced-concrete slab less than a foot below the underside of sleeper.

Figs 2


Ring canal
Church lane
New rail-level
Amsterdam datum
KEY
Clay
Peat
Sand
Topsoil
Depth of excavation
Chapel road
State highway No. 3
NATURE OF GROUND



ROTTERDAM–NIEUWERKERK. EXTRACT FROM LONGITUDINAL SECTION

Sand was first deposited, on both sides, at the bottom of the embankment to act as a toe-counterweight in order to prevent the embankment slipping further. A temporary track was then laid on one of these sand " dams " to enable complete possession of one road to be obtained.

The old subgrade was scraped away and beach-ballast deposited between wooden shutter-boards about 11 feet apart and to a depth of 16 inches, a light steel mesh being incorporated near the bottom of this layer, which was then concreted by pumping-in cement and sand grout to produce " no-fines " concrete (*Figs 3*). When this concrete had set, 7 inches of old ballast was spread on top of it and consolidated by 3-ton vibrating-plate-type compaction machines. Finally, 4 inches of new ballast was put down under the sleepers and consolidated by a Matisa tamper running over the new track.

Boxtel—Best

The work at this site was primarily a re-laying operation, involving a change-over to the new heavy-section (132 lb. per yard) rail, although advantage was also taken of the opportunity to strengthen and re-ballast the road-bed.

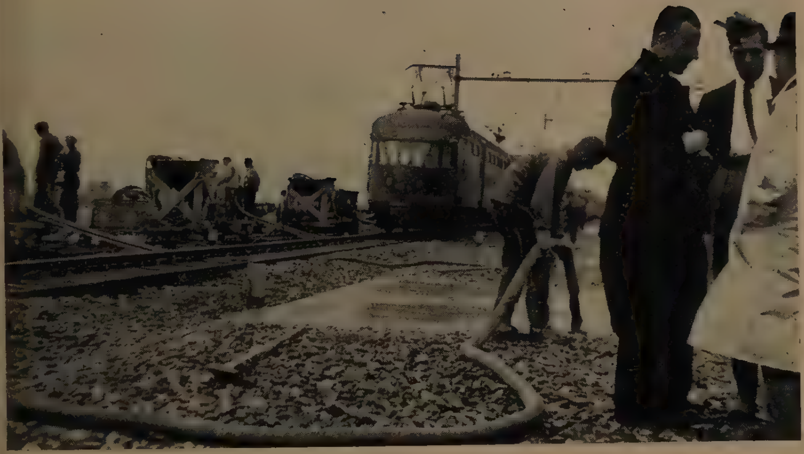
Since possessions were difficult to obtain between these two stations, a temporary track was laid on one side of the existing double-track line ; the overhead conductors for it were supported on temporary cantilever brackets bolted to the existing portal frames, and a new drainage ditch was cut outside the old one, over which sand ballast for the temporary track was laid. Since the line is on a low embankment at this place, and the soil is sandy, no extra drainage was required apart from ensuring that the relatively impermeable layer of grass on the verges of the old ditch was removed (see *Fig. 4*).

The running lines were then re-laid in turn. After lifting the old track, the old ballast was swept to one side to a depth of 18 inches with a ballast plough, the sub-grade consolidated with 3-ton vibrating-plate compactors, and new broken-stone ballast was laid under the new track and compacted in 3-inch layers by a Matisa tamper.

Equipment used in Track Stabilizing Work

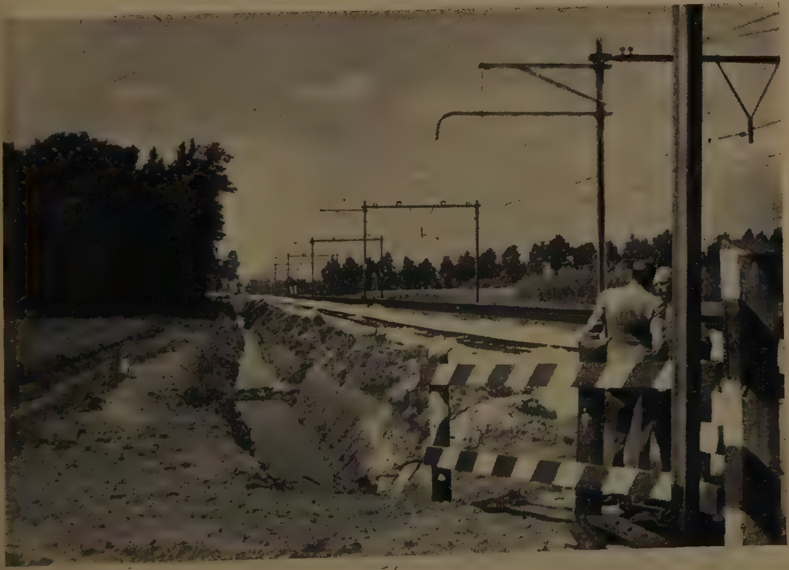
In addition to the ordinary civil engineering machinery such as draglines, bulldozers, and compressed-air tools, the Netherlands Railways also make considerable use of specialized machinery. The 3-ton vibrating-plate machines are owned by the Railways and hired out to contractors for particular jobs. A number of Matisa tampers similar to those used in Great Britain are also available, and are used for fettling whenever a re-ballasting operation is carried out. It is interesting to note that these machines are operated by a special mobile re-laying gang which takes over prior to re-opening in order to carry out the final fettling.

Fig. 3



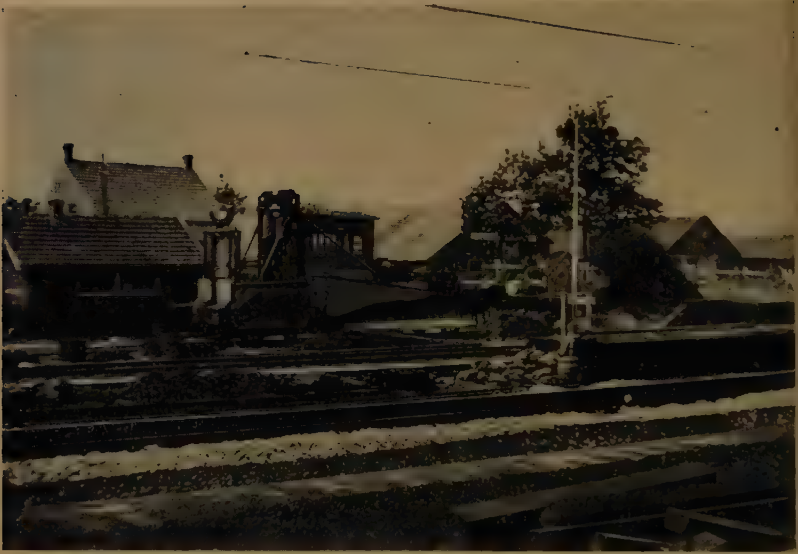
Gouda–Oudewater. Producing "No-Fines" Concrete Slab by Grouting

Fig. 4



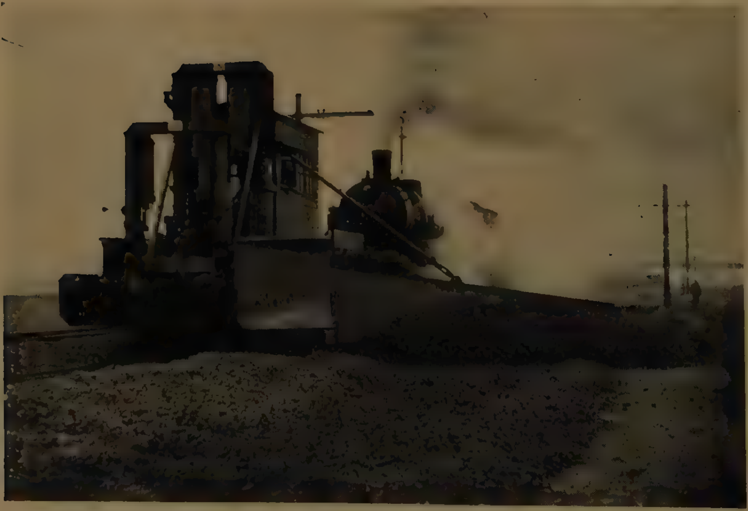
Boxtel–Best. Temporary Track Laid on Sand Ballast over Line of Old Ditch

Fig. 5



Ballast Plough. Blades in Travelling Position

Fig. 6



Ballast Plough. Longer Blade only in Use

Another machine used is the ballast plough. This consists of a four-wheeled unsprung wagon of long wheelbase with two vertically-hinged blades on each side, which can be raised or lowered, pushed out or in, or tilted by link-bars worked by compressed air (*Fig. 5*). When operating, the plough is propelled over one road with either or both blades extended over the road to be swept. It is able to make a cut of up to 10 inches in one pass through ballast, sweeping it into the far cess (*Fig. 6*) The surface produced is remarkably even and a skilled operator is able to produce a very uniform level far more efficiently than with a bulldozer. The disadvantages of the use of this machine are, of course, that two adjacent roads are required, and the material swept aside has normally to be hand-loaded for removal.

A machine seen in use on another site was a small bulldozer fitted with a scoop-blade which was able to pick up dirty ballast from the track and load it directly into dumpers. The chief advantage of this machine is that it is able, in suitable circumstances, to do the work of a normal bulldozer and a grab or dragline for small amounts of excavation and it is much more easily handled close to the track.

ELECTRIFICATION

General

Following on the reconstruction of a large part of the railway system due to extensive damage during the war, a considerable route mileage was scheduled for electrification, in addition to that already electrified before the war.

Statistics show that 50 per cent of the annual revenue of the Railway is obtained from 5 per cent of the route-mileage, and 95 per cent of the revenue from 30 per cent of the mileage, so that the lines economically suitable for electrification were easily selected.

All electrification on the Netherlands Railways is operated on the overhead system at 1,500 volts direct current.

Types of Overhead Portals

Various types of portals were erected before the war, many of them in steel. Owing to the scarcity of steel in the post-war period, the use of reinforced concrete portals was developed when electrification was recommenced. The first type used, consisted of spun reinforced-concrete masts with a bow-shaped concrete cross-member, reinforced with an external straight steel tie-bar. Recently, pre-stressed concrete masts and cross-members have been erected at a considerable saving of expense. Terminal portals in pre-stressed concrete have also been used, but owing to the number and variety of attachments involved, as well as the extra strength required, it has now been decided that these shall, in future, be made of galvanized broad-flanged steel beams. For junctions, and places where the span is not standard, steel portals, braced as necessary, are being used; catenary suspension cables between steel pylons are employed for large spans.

Fig. 7 shows an intermediate portal in pre-stressed concrete constructed on the Hoyer system. In the pre-stressed masts, the diagonal web-bracing is reinforced with a cage of unstressed reinforcement, spot-welded for ease of placing, the pre-stressed wires being located in the flanges.

Construction of Foundations for Overhead Portals

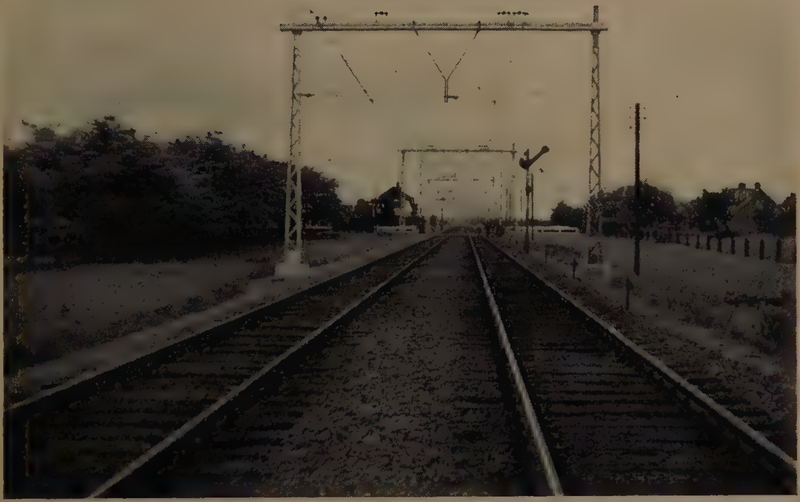
Since long lengths of track in the Netherlands are carried on low embankments with cycle paths on each side, it has been possible to support the vertical masts in standardized pre-cast concrete bases which are connected together under the track, where the subgrade is weak, by old rail ties. These foundation-blocks and the anchor-blocks for the terminal portals are pre-cast on a production-line basis at a depot situated centrally on the route which is being electrified (*Fig. 8*). Sections of timber shuttering bolted together are used for casting the lower part, square in plan, of these foundation blocks; split elliptical steel shuttering is used for the upper part, the cores of which are also steel, cone-shaped for easy withdrawal. After waterproof paper has been laid on the platform to prevent adhesion, the moulds are erected, and the reinforcement placed in position, including the short length of old rail to which the cross-tie is subsequently attached. Fairly dry concrete made with rapid-hardening cement is then poured in and consolidated with small hand-vibrators. After one hour the top core is removed by sheer legs (see *Fig. 8*), and after four hours the outside steel shuttering is also removed, thus enabling it to be re-used every day. A light steel gantry carrying a skip is attached to the main mixer, the whole erection being mounted on wheels so that it can travel along the production line.

Erection of Portals

A special work-train has been made up for the erection of the portals. It places the foundation and anchor-blocks and then erects the superstructure.

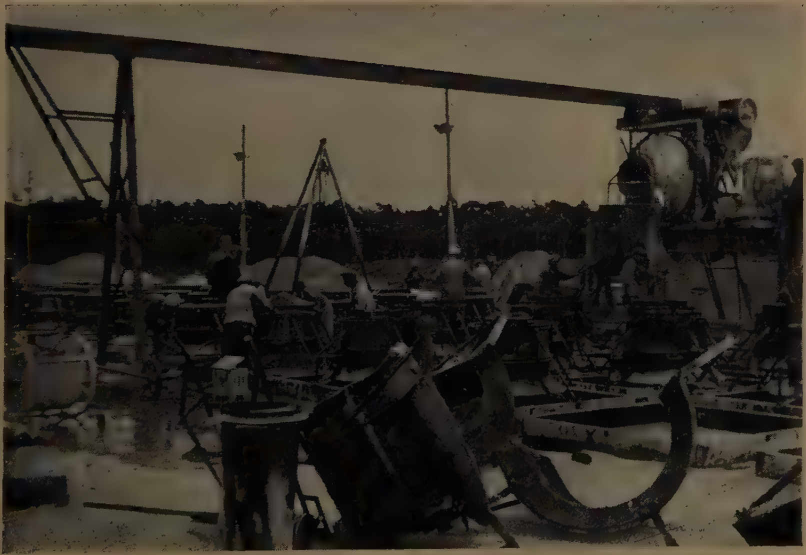
This train comprises a small rail-mounted crane with special balance-rollers in order to avoid fouling the other road while the jib is over the cess of the road on which the train is standing, and a number of flat wagons on which a rail track is laid which can be temporarily connected over the buffers by further short lengths of rail. For block-placing, which is carried out by running the work-train through during intervals in normal traffic, the blocks are loaded on the flat wagons, jacked up one by one and wheeled on a small trolley up to the crane which places them in previously excavated holes (see *Figs 9* and *10*).

Fig. 7



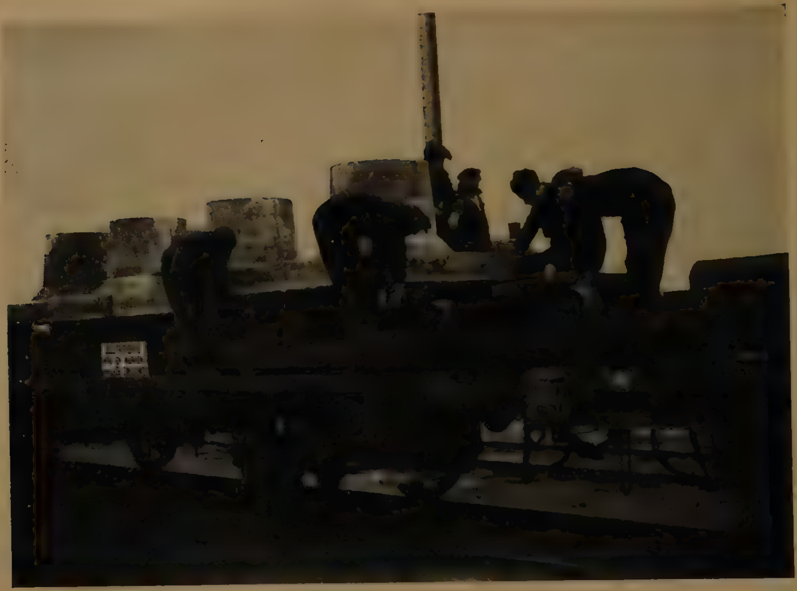
Pre-stressed Concrete Intermediate Portals

Fig. 8



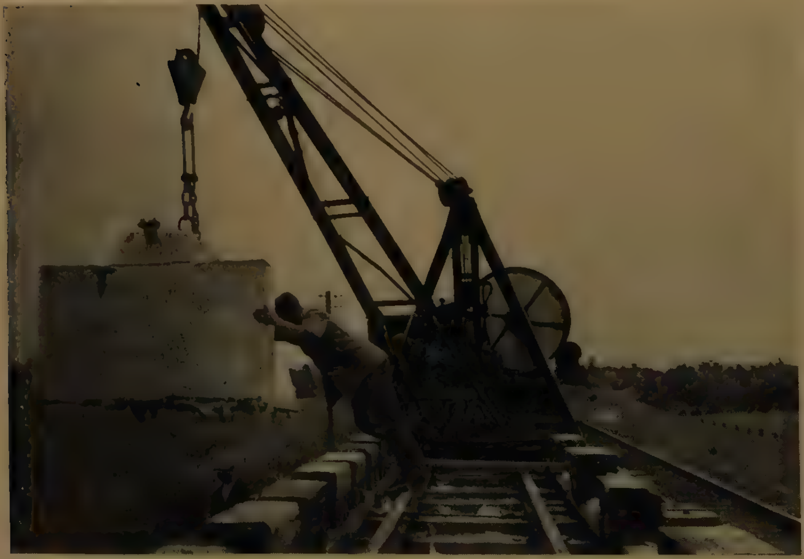
Legerplaats–Oldebroek. Pre-cast Concrete Production Depot

Fig. 9



Electrification Work-Train. Concrete Foundation-Block Being Placed on Trolley for Moving to Crane

Fig. 10



Electrification Work-Train. Anchor Block being Swung into Position

ACKNOWLEDGEMENTS

The Author wishes to record his respectful thanks to Mr F. Q. den Hollander, President, Netherlands Railways, for his kindness and for his permission to publish the material contained in the Note, and also to Mr J. L. A. Cuperus, Chief of Way and Works, and all the other officers of the Netherlands Railways he met, for their hospitality, consideration, and help to him on his visit.

The Author wishes also to thank Mr J. C. L. Train, M.C., M.I.C.E., Member of the Railway Executive, for his permission to present the Note, and Mr V. A. M. Robertson, C.B.E., M.C., Past-President I.C.E., lately Chief Civil Engineer, British Railways, Southern Region, and Mr J. I. Campbell, M.I.C.E., Civil Engineer, Eastern Region, for the guidance and assistance given to him in preparing it.

The Note is accompanied by thirteen photographs and one sheet of diagrams, from some of which the half-tone page plates and the Figures in the text have been prepared.

Introductory Note No. 2

"Observations on a Visit to the French Railways in 1949"

by

ROBERT GEOFFREY THURTLE

SYNOPSIS

The Note briefly describes the organization of the Engineering Department of a Region of French Railways, and some details are given of their standard plain line and crossing work.

The basis on which permanent way maintenance is scheduled and carried out is given, and a typical method used for correcting a standard track defect is described.

Reference is made to the "Mauzin" track record which has superseded the "Hallade" record in France, and gives a more comprehensive picture of track condition.

Hand and mechanical methods used for permanent way re-laying are referred to, and a method is described for re-laying pre-assembled track which involves the occupation of only one line.

Under the heading of "Mechanical Plant" the Author briefly describes a mechanized ballast cleaner which automatically raises the track for the passage of the buckets.

INTRODUCTION

IN the summer of 1949 the Author was attached, for 6 weeks, to the Civil Engineering Department of French Railways, and was given facilities during this time to see the general organization and work of the department in the Ouest Region.

The knowledge obtained, which is the basis of this Introductory Note, is limited to this Region, and therefore it is not possible to give more than a brief outline of the French organization, with a description of the more interesting of their maintenance methods.

ORGANIZATION

General

The Region in French Railways has three working departments. Two of these are responsible for Motive Power, Rolling Stock, and Stores, and Operating and Commercial matters respectively, and the third is the Civil Engineering Department. The responsibilities of this department in their organization are therefore wider than those of its counter-part in Great Britain, and cover Signal and Telegraph, Estate, and Electrical and Outdoor Machinery matters.

Engineering Department

The Regional Civil Engineer has a head office with a staff of about five hundred, and District Engineers are responsible to him for areas served by about 900 miles of single track.

The Engineering District is divided into about six Sections, each in the charge of an Engineer who has a staff consisting of an Inspector, who deals with Signalling and Electrical matters, and about three clerks. The Section Engineer supervises about five Inspectors' Districts.

An Inspector's District consists of about four Track Gang Lengths, each gang consisting of about nine men.

TRACK

Plain Line

The standard track in France consists of flat-bottomed rails fastened to hardwood sleepers by coach-screws—three screws per sleeper, per rail, two inside and outside alternately. The rails are fixed at an angle of 1 in 20 to the vertical by adzing the sleepers, even where base-plates are used, which is now the practice on main lines. The sleepers are of oak or beech, and are unsquared. They are creosoted, and often bound at the ends with a steel strip to minimise splitting. The standard rail weights are now 46 and 50 kilograms per metre, equivalent to about 93 and 102 lb. per yard, respectively. Special rails of 55 kilograms per metre, or about 111 lb. per yard, are used in tunnels. Normal rail lengths are 18 metres or about 59 feet. Ballast is generally of stone, perhaps slightly larger in size and more generous in quantity than is normal in Great Britain.

Crossing Work

Switch and Crossing work is completely standardized. For main-line fittings, standard crossings are cast in manganese steel, and the guard rails to these, which are of a special section, are fastened to the timbers independently of the running rails, and maintained at the correct distance from the crossing-nose by means of a steel strut across the fourfoot, which is coach-screwed to the timber. This sound method has no doubt been evolved in view of the tendency for the hardwood to shrink and so tighten the gauge. Standard switches are flexible and are made from specially rolled switch- and stockrail sections which give the tilt to the head, whilst the bottom flange rests on level slide-plates. This avoids any twisting of the rails in manufacture or use. To avoid any possibility of movement at the heel-joint, a portion of the bottom flange of the switch-rail, which has a very thick web, is cut away for a short length, about two timbers from the joint.

The lighter section rail is used for fittings in sidings, and the crossings for these are fabricated from rail as in Great Britain.

Squared timbers are used for crossing-work, and this is ballasted with one-inch chippings.

PERMANENT WAY MAINTENANCE

Day-to-day maintenance is carried out by railway personnel in accordance with a programme prepared annually for his district by the Inspector. This programme takes into account an annual calendar of work which specifies what is to be done each week, and includes the " complete repair " of one-quarter of the route length, and the " partial repair " of a further quarter, during each year.

A " complete repair " consists of the replacement of defective sleepers and the turning or replacement of worn rails if necessary, the dismantling, cleaning, and inspection of fishplates, with replacement by reconditioned ones where required ; the pulling back of rails as needed ; cleaning of ballast, refixing and tightening of coach-screws, followed by the slewing of the track to alignment, and measured shovel packing throughout to give a lift of about a ¼ inch.

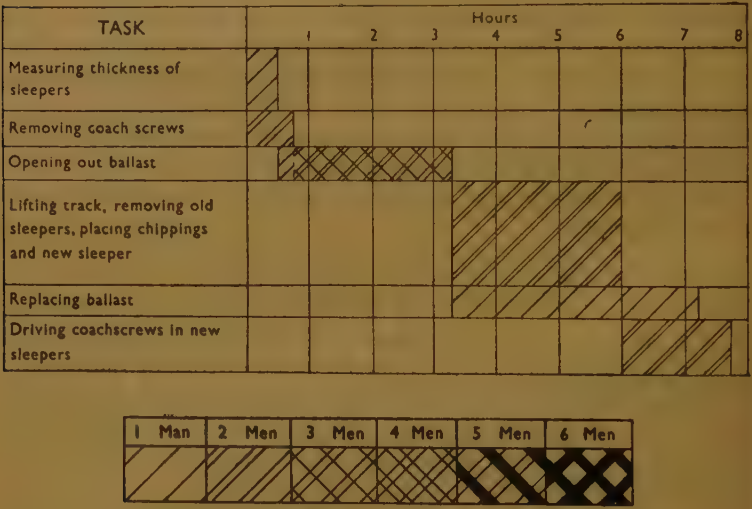
A " partial repair " is a review of the " top " of the track, measured shovel packing being carried out at joints and slack places, but any component which will not last 2 years to the next " complete repair " is replaced at this time.

Included in the calendar, but for the whole length of route, is the oiling of fishplates and pulling back of rails, tightening of coach-screws, and the measured shovel packing of joints.

When the programme has been prepared by the Inspector, it is sent via the section to the District Engineer who adjusts the distribution of labour as necessary. Gangs are usually grouped together on lengths due for " complete or partial repair," to provide enough men for this work and to simplify supervision.

The programme includes all other work done by the gang, such as weeding, grass cutting, etc. It shows the location of work and accounts for all manhours available during the year. Programming in this detail is possible only because, for each operation in day-to-day maintenance, work-tables have been prepared for gangs of almost all sizes, which show the normal output to be expected, and details of each man's task throughout the day (see work tables for changing defective sleepers in *Figs 11*, *12*, and *13*). Under this system each operation has been broken down into simple tasks, and each man of the gang does one such task for almost the whole day, carrying with him the one or two tools he needs. The advantages claimed for this production-line system are that supervision is easy, no time is lost in fetching tools, and it is not possible for an idler to do less than his share of work.

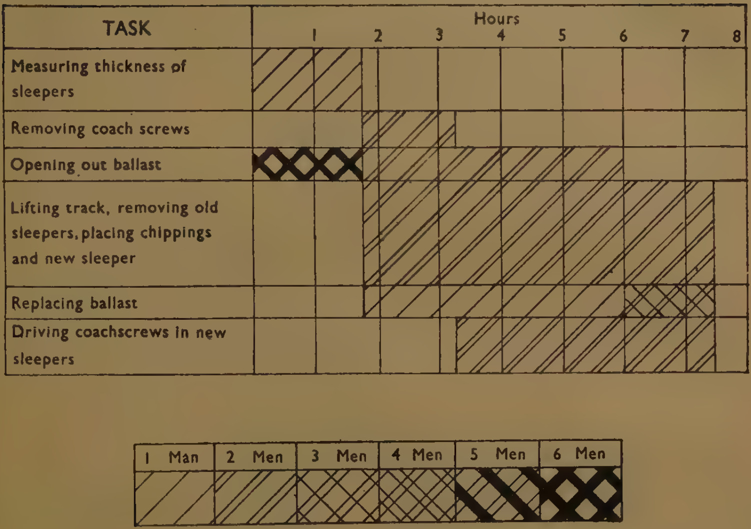
Figs 11



3-Man Gang, Output 30 Sleepers

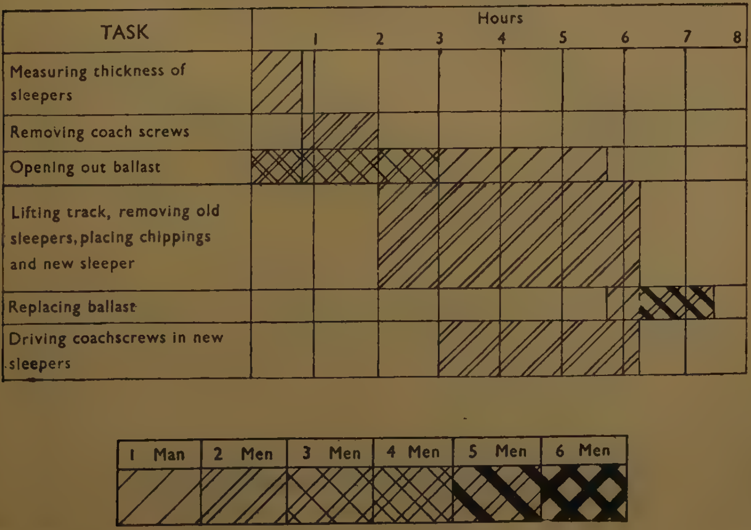
Close study has also been given to all tools, and where necessary these have been specially designed for specific purposes. An interesting example of such a tool is the " Lemaire " shovel for use in measured shovel packing (*Figs 14*). This tool enables chippings to be placed under a sleeper without the operator bending his back, and requires the minimum of opening out of ballast. Typical of the care given to details, is the fact that this tool can be modified in a matter of minutes to enable it to be used by a left-handed operator.

Figs 12



5-Man Gang, Output 50 Sleepers

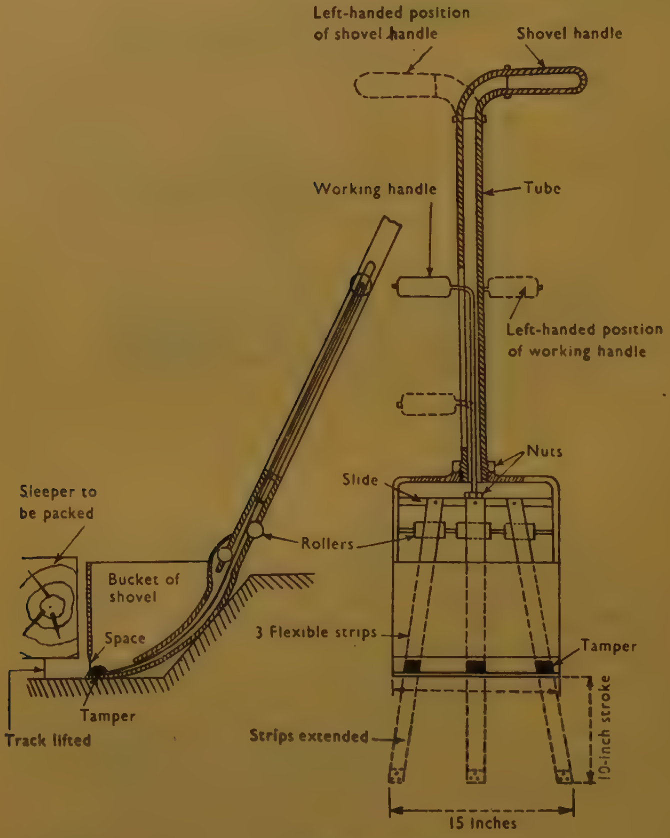
Figs 13



7-Man Gang, Output 70 Sleepers

In addition to hand tools, mechanized tools have been specially designed, and are provided as standard equipment for carrying out the annual maintenance work in certain areas. One of the jobs the Author was able to see being done by both hand and mechanical means was the

Figs 14



" LEMAIRE " SHOVEL

tightening of coach-screws as part of the " complete repair " of a length of track laid without base-plates.

Many lines have no base-plates, and in time, despite the hard wood of the sleepers the rail wears into them to a point where the coach-screws cannot grip it. When a length of track approaches this condition, the shoulders of the sleeper adjoining the bottom of the rail are removed during " complete repair " to enable the coach-screws to be re-tightened.

The hand method of doing this, involving the use of chisels and adzes, has been largely superseded by the mechanized method which employs the following specially designed tools:

(*a*) a light petrol-driven coach-screw driver mounted on rollers over one rail and balanced by a light channel to a roller running on the other rail;

(*b*) an electric milling-machine mounted on diplories and capable of removing the timber alongside the bottom flange of the rail to a predetermined depth;

(*c*) an electrically-operated coach-screw driver similarly mounted to the petrol-driven one; and

(*d*) an electrical generator specially mounted on diplories in such a way that the generator and trolley could be removed from the track, whether in a cutting or on an embankment, in less than a minute.

The following is a typical example of the "production-line method" usually adopted. Coach-screws were unscrewed on the outside of the rail by one man with the petrol machine, another man removed the fishplates and all the loosened coach-screws, then followed the milling machine operated by one man, and the surfaces of the timber thus exposed were creosoted by yet another man. The next man oiled the coachscrews and replaced them in the holes, into which they were driven by the following man with the electric coach-screwdriver. The generator and operator followed, and behind that was a precisely similar gang dealing with the inside of the same rail.

Such gangs as these, which do special work, are often housed in trains which are parked in stations near the site. The train consists of sleeping and messing coaches with kitchen and ice-box, and includes such amenities as hot and cold showers, library, and radio. The men in these gangs are paid above the standard rate, and also receive extra pay while living away from home, but taking this into account, the output is found to show a gain of about 20 per cent over other methods.

Almost all curves are provided with "monuments." On the Ouest region, many of these took the form of concrete blocks having timber inserts in the tops. Nails were driven into these during "complete repair" whenever necessary to give a new alignment, which was made by stringline methods.

Welding is used for repairs to switches and crossings which are built up by the electric-arc process, and a special technique has been developed for successfully treating the manganese crossings in this way. The thermit process is also widely used for making joints in rails. In sidings and marshalling yards, almost all joints are treated in this way to cut down maintenance work, giving rail lengths of more than 200 yards. The Author also saw this method being used to join pairs of short rails for use in running lines. A specially fitted wagon was available for the work of producing moulds, and short lengths of rail of all types were available as patterns. No difficulty was experienced in joining rails of different section by this means. In the case of the short rails being joined, pre-heating of the rail ends was being tried in an attempt to improve the joint.

" Mauzin " Track Records

The " Mauzin " carriage has now superseded the " Hallade " recorder in assessing the condition of the track. It consists of a normal-type bogie-carriage with an additional special bogie in the centre. From this bogie, pens are operated to indicate on a roll of paper the gauge, curvature, super elevations, and line and level of each rail. The roll of paper runs at a rate proportional to the speed of the vehicle so that 20 centimetres on the record represents 1 kilometre of track. This is different from the Hallade recorder where the paper moves at a constant speed. It has the advantage of making it easy to locate on the ground, defects shown on the record, and avoids the necessity of writing-up before making copies. The carriage in which the District Engineer, Section Engineers, and Inspectors ride, is normally run at the rear of the train. The previous record can be run simultaneously enabling comparisons to be made there and then.

Permanent Way Re-laying

This work is done in France by contractors. The Railway provides and delivers all material, and railway employees are responsible for traffic security arrangements. The contractor normally has the choice between hand or mechanical methods, and this depends to some extent on the location of the work. Whenever possible, all other work such as cleaning and renewal of ballast, renewal of level crossings, and re-alignment of curves is carried out at the same time as the re-laying of the length of track concerned. Following re-laying, the track is maintained for 6 weeks by the contractors.

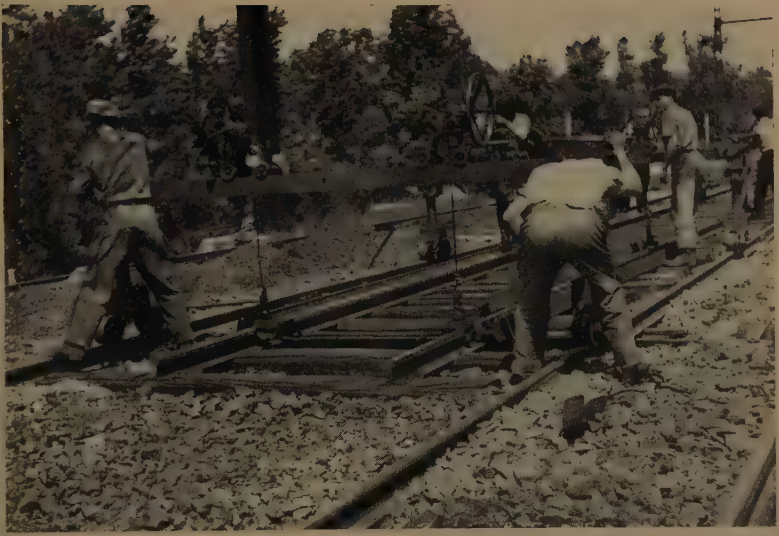
The Author was able to see the following methods of track re-laying in progress :

(1) A hand method, with maximum preparation in advance, that is to say, all ballast cleared out from between sleepers, and all new materials tipped on the site before breaking the road.

(2) A completely mechanized method, using a ballast cleaner and a self-propelled twin-jibbed crane running on the opposite line and lifting out old sections of track and replacing it with new pre-assembled track.

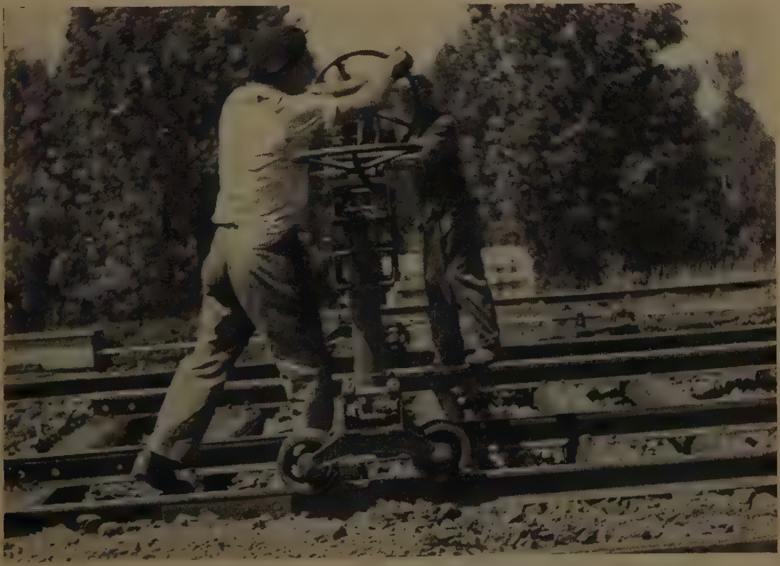
Both these methods were not fundamentally different from those employed in Great Britain.

Fig. 15



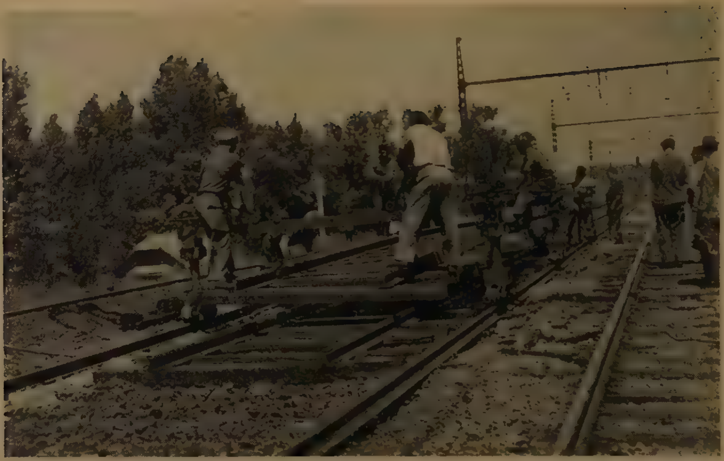
Preparing to Lift a Length of Old Track, using Hand Portals

Fig. 16



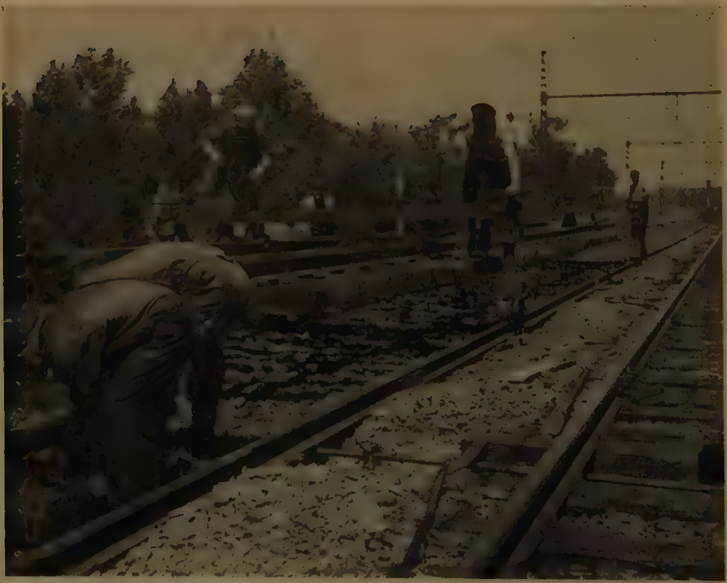
Lifting Old Track, showing Adjustable End Pillar of Portal

Fig. 17



Transferring Lifted Length to Diplories

Fig. 18



New Length about to be Lowered into Position

(3) Another completely mechanized method which included ballast cleaning, but in place of the twin-jibbed crane, hand-propelled portals were used to lift out sections of old track and replace them by pre-assembled lengths brought to the site on diplories.

The hand-operated portals (*Figs 15*, *16*, *17*, and *18*) were developed from roller-bearing diplories and span the track to be relaid. They run on light-section flat-bottom rails temporarily laid on wooden blocks at about 10-foot centres outside the sleeper ends. The following method is used. Pre-assembled track is brought to a siding in a station yard adjacent to the work. Here the lengths are transferred to diplories (see *Fig. 19*) by means of a temporary gantry erected over the wagon and the track on which they run. Each pair of diplories is loaded with two lengths of track, one on top of the other, and successive lengths are formed into a train which is pushed to the site of work by a motor-trolley. At the site of the work, two lengths of old track are lifted out by the hand-portals and loaded on a pair of diplories standing on the track to be renewed. Simultaneously, lengths of new track are lifted in by another set of portals. The trains of new and old track are moved along with the work, and diplories are man-handled over the gap between new and old tracks to receive the next length of old track. In this way only one track is occupied.

This great advantage is exploited to the full by the French, and the Author was given details of the scheme adopted by the re-laying of about 52 kilometres (32¼ miles) of the main line from Bordeaux to Angouleme, which made use of these hand portals in conjunction with mechanical ballast-cleaning and ballast-tamping. This line carries about 120 regular trains a day, and to allow work to proceed without interruptions for the whole day, special single-line working sections were established. Where necessary, facing crossovers were specially laid in the track to be re-laid, and during the day all traffic was diverted to the opposite track which was used for single-line working as far as the next morning trailing crossover. Special signalling was installed to allow this to be done without the necessity for trains in either direction to stop. Trains running on their normal track were not, in fact, restricted in speed at all, and those running on the "wrong" road were restricted to about 40 miles per hour. From six o'clock in the evening until eight next morning, single-line working was given up, and the only restriction of speed, of about 19 miles per hour applied to the actual length of track being worked on and did not normally exceed 1 mile in length. The work progressed towards Paris at about 500 metres, or 25 chains, each working day. This basic rate is determined by the speed of the mechanical ballast-cleaner in view of the method of work adopted. When the work was in full swing, the ballast cleaner was at the Paris end of the work and worked towards Paris all day. Behind it, the track which had had the ballast cleaned the previous day was re-laid by means of the portals with pre-assembled new track; behind this, new ballast was tipped on the track laid the previous day and the track was lifted and tamped with the Matisa Tamper. Before ceasing work for the day, the temporary track for the portals was carried forward and laid alongside the track where the ballast had been cleaned, ready for the following day's re-laying. When single-line working was given up each night and on Sunday, the ideal position would be that about 25 chains of track would have had the ballast cleaned, a similar length would have been re-laid, whilst a further 25 chains would be in process of final lifting, lining, and tamping. On either side of this length of about a mile, there would be untouched old track or completed new track. Where permanent way fittings occur, and alongside platforms, it is not possible to use either the mechanical ballast-cleaner or the land portals, and in an operation such as the one described, such lengths would need to be dealt with specially. Special temporary local depots were actually arranged for providing the new track and dealing with the old, in the scheme described above, but in a smaller scheme these might not be necessary.

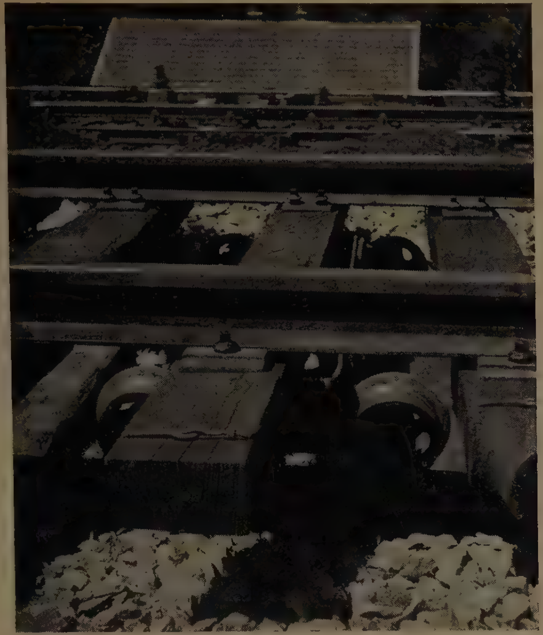
Mechanical Plant

The French railways are well equipped with small mechanical tools. Mechanized coach-screw-drivers, rail-saws, and drills are widely used for maintenance work. Pneumatic hand tamping-hammers had been tried, but were not in general use, owing, no doubt, to the universal adoption of measured shovel packing. The Matisa mechanical ballast-tamper is used extensively in France, but normally these machines, together with all other heavy plant, are the property of the contractors. The Author saw three different types of mechanical ballast-cleaners in use. They were the Matisa machine which is being used in Great Britain, a Swiss machine, which was self-propelled, but required the track to be raised on to the timber blocks while in use, and a machine known as the S.E.C.O. ballast cleaner. This consists, essentially, of a girder, carrying riddles, and mounted on bogies at about 100-foot centres, which raises the track between the bogies to allow the passage of the buckets. This machine winched itself forward when in use, to a fixed point provided by a locomotive, which ran forward as necessary (see *Figs 20* and *21*).

War Damage Repairs

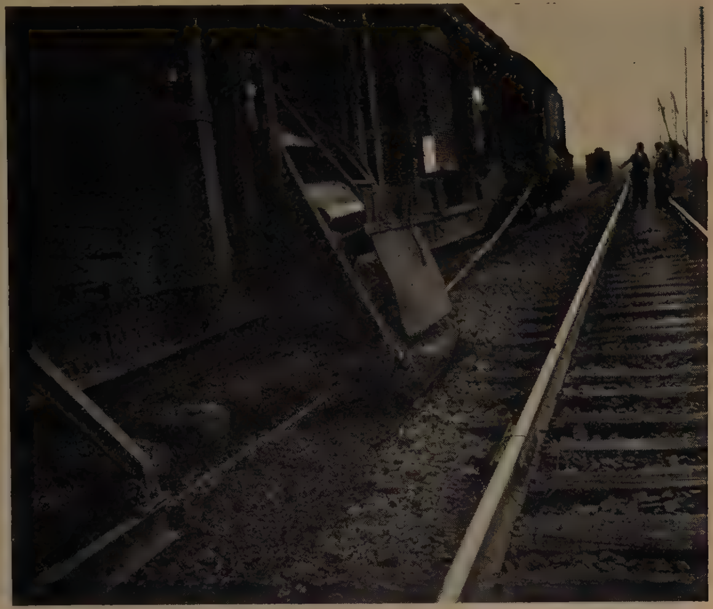
When the Author was there, the majority of repairs to bridges, marshalling yards, and locomotive depots, on the main routes in France had been completed. It was clear, however, that an immense amount of work had been done. In almost all cases, the new designs for facilities replacing those destroyed incorporated improvements. The new marshalling yards with their electrical wagon-routing, loudspeaker systems, and central

Fig. 19



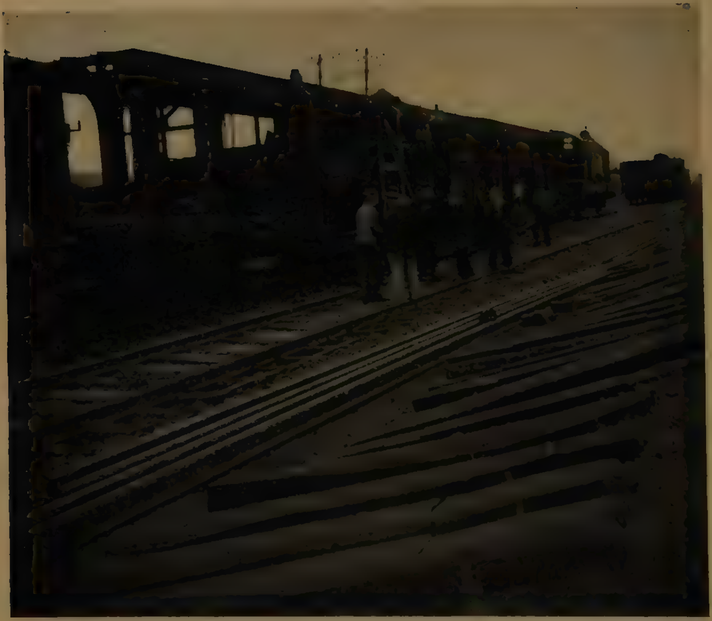
Close View of a New Length Seated on Special Diplories for Transport to Site

Fig. 20



S.E.C.O. Ballast Cleaner, showing Track Automatically Lifted for Passage of Buckets

Fig. 21



S.E.C.O. Ballast Cleaner and Train

amenities buildings were of a very high standard, and must be among the most modern in existence.

The terrible damage suffered by the French Railways during the war has provided an opportunity, which has been seized and used with imagination, to carry out extensive modernization which is bound to be an asset for the future.

General Impressions

The Author was impressed by the detail in which all engineering department work was organized. All maintenance operations had been broken down to elementary tasks, for which simple but effective tools were available. Booklets issued to supervisory staff detailed standard methods for all work.

Many factors contribute to the close control of work obtained. One of these is undoubtedly the additional supervision given by the introduction of Section Engineers between the Inspectors and District Engineers. Another is the method adopted for the training of engineers.

The first railway appointment of a young man from an Engineering School, following a course of three months at the Regional Railway College would be as an Inspector in charge of track gangs. In the Author's opinion, this close application of trained minds to day-to-day track work, combined with the early knowledge of handling staff which this system provides, is probably the fundamental reason for the effective organization of work which exists on the French Railways.

A great deal of attention is given to staff matters at all levels, and the fact that the highest administrative posts are held by qualified engineers strengthens the position of the Engineering departments.

A bonus is paid to all employees in relation to the amount and quality of the work done, and marks are awarded individually for merit and ability which influence the amount of bonus received, and affect promotion, which is by selection on merit.

Courses of instruction are held for all grades of supervisors in the Region College, where they are residents for the period of their course. This gives an opportunity for the exchange of views, allows new standard methods to be effectively propagated, and enables members taking the courses to be assessed by a uniform standard.

These measures, plus the advantages of railway work compared with other employment in France, are probably the reasons for the universal keenness and enthusiasm of the average French railwayman for his work which was, apart from the kindness with which he was received, the Author's most striking impression of his visit.

ACKNOWLEDGEMENTS

The Author wishes to express his thanks to Mr J. C. L. Train, M.C., M.I.C.E., Member of the Railway Executive and Mr M. G. R. Smith, M.B.E., B.Sc., M.I.C.E., Civil Engineer, British Railways, Western Region, for permission to present the Note and for help with its preparation.

The Note is accompanied by seven photographs and four sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

The Introducers opened the discussion by showing a series of lantern slides.

Mr J. G. F. Inglis said that he proposed to describe some features of Dutch practice additional to those dealt with in his Note.

He had been asked, first of all, to give a few figures to show the relative size of the Netherlands Railways. For comparative purposes he had taken the figures for the Southern and Western Regions of British Railways, making the figures for the Western Region 100 in each case. On that basis the figures were as follows:

Total route miles: Western Region, 100; Southern Region, 60; Netherlands Railways, 53. Of this mileage, 32 per cent of the Southern Region and 28 per cent of the Netherlands Railways were electrified.

Steam locomotives: Western Region, 100; Southern Region, 49; Netherlands Railways, 19.

Total passenger carriages: Western Region, 100; Southern Region, 108 (51 per cent in electric stock); Netherlands Railways, 63 (63 per cent in multiple-unit electric stock and 17 per cent in multiple-unit Diesel stock).

Wagons: Western Region, 100; Southern Region, 41; Netherlands Railways, 24; that last item had to be treated with caution if taken as a yardstick for the volume of freight traffic handled, because Mr Inglis believed that the turn-round of wagons in the Netherlands was two or three times quicker than in Great Britain, partly owing to the much shorter average hauls.

Engineering districts: since there were twelve in the Western Region, six in the Southern Region, and eight for the Netherlands Railways, it might be taken that the Netherlands districts were somewhat smaller on the average than those in Great Britain.

The Chairman remarked that the Introducers had shown the value of going abroad and seeing how other people did things. There were many who, like himself, had believed in doing that, and if further proof of its value were wanted it had been given that evening. The Introducers were to be congratulated on having noticed so much during their visits.

There were one or two items which he would like to mention. Mr Inglis had referred to the new railway which was being constructed in the neighbourhood of Rotterdam, and the Chairman happened to have seen the work which was being done there. It was startling to read that the engineers there had dug down 15 feet before they considered that they had reached a stable foundation and that they had then filled in with sand and built an embankment on top of that.

The remarks on p. 389 with regard to the overhead portals carrying the overhead wire were very interesting. He had seen some of them, and those made in pre-stressed concrete were of great interest. Mr Inglis had said enough to show that the Netherlands Railways were extraordinarily efficient, and the Chairman believed that they were one of the few railway systems in the world which had managed to operate with a credit balance. Mr den Hollander, the President of the Netherlands Railways, and Mr Cuperus, the Chief Engineer, had been kind enough to come to London on Saturday last for a Dinner of the Permanent Way Institution; they had also been kind enough to invite the members of that Institution to visit Holland, and had entertained about three hundred members. Those who had taken part in that visit were familiar with some of the things shown in Mr Inglis's slides, which were most interesting.

With regard to Mr Thurtle's contribution, the Chairman was glad that on p. 398 the Mauzin machine was referred to. It might be of interest to mention that the President of the Institution, whom they were honoured to have present that evening, had gone with him to Belgium about 2 years ago to see that machine. If it had been possible to construct one more readily to the British gauge, a trial might have been made of it here. It was hoped within a short time to have one of the machines on trial in Great Britain.

The Chairman had also been interested to note, from what had been said on the same page, that contractors were employed so freely on the French Railways, because with present labour conditions they were being driven to employ contractors on British Railways, whereas presumably the French did it voluntarily. The method of track renewal referred to on p. 399 was, he believed, being tried out on the Eastern Region, and perhaps someone who took part in the discussion would say something about that.

The French, it would be agreed, were extraordinarily logical people, and it was well worth studying their methods, as Mr Thurtle had done so ably. At the same time, if British Railways were to incur the financial loss suffered annually by the French Railways, a loss which he believed was more than £100,000,000, the Railway Executive would come in for even more adverse criticism than it received already. That, however, did not detract from the efficiency of the French Railways, because he imagined that their experts would do a good deal more in the way of economizing if they were not over-ruled politically, and there was no reason for not admiring their methods. An important point to remember, however, was that the density of traffic on the French Railways was nothing like so great as that on the British Railways. He believed that the density of traffic on British Railways was greater than that on any other railway system.

Mr M. G. R. Smith observed that on p. 389 Mr Inglis had referred to a small bulldozer fitted with a scoop-blade, which was able to pick up dirty ballast from the track and load it directly into dumpers. Mr Smith was not clear from that description exactly what type of machine Mr Inglis had in mind, and requested some further description of it.

Mr Thurtle had referred to the measured packing equipment used in France. It was of particular interest that the French practised measured packing very extensively and used an entirely different set of equipment from that used in Great Britain. Mr Smith believed that at one time the French had used equipment supplied by Great Britain, but had then changed over to a quite different type. The latter had been given a good trial in Great Britain recently, but it had been decided that British equipment was more suitable for British Railways' purposes.

Another thing of interest which he had seen in France was the loading up of dirty ballast from the ballast-cleaners—some the same as were used in Great Britain and some different—into wagons standing behind the ballast-cleaner and therefore occupying one road only, which was a great help in getting increased use out of those ballast-cleaners.

The void-meter used in France was pressed down by hand onto a sleeper ; the points of its three legs rested in the ballast, and it was not necessary to drive anything into the ground. When the sleeper was depressed a spring pushed a sliding collar up the central pillar of the meter and the voids were read in the distance between the sliding collar and the top of the main frame of the meter. It had the disadvantage that it was necessary to look sideways to see what the reading was, and there was a possibility that the collar might slip down a little, but it was very easy to fit up. A telescope was used with a graduated scale as sighting board to read the depressions in the rail-top level and it could also be used to read the versines on curves by reversing the scale. The disadvantage of the apparatus was that it depended entirely on its being absolutely vertical, and there was a chance that there might be a slight tilt on either the telescope or the scale, so that the depression readings or the versine readings would be incorrect. An elevator raised the dirty ballast into the hopper fitted on the end wagon, and a dumper ran backwards and forwards along the wagons. The dumper was filled from the hopper and it dumped the material in the further wagon, working backwards and forwards until it had filled the whole rake of wagons. He thought that that method could very well be developed for use in Great Britain. On reaching a bridge it would be necessary to lower the hopper, which would be a nuisance, but the number of occasions would be small in relation to the value of such an arrangement and the increased use which could be got out of those machines.

Mr J. B. Manson observed that in the Nord Region in France, to which he had been attached in 1950, what had struck him most was that all grades of French railwaymen that he met were intensely interested in their work. That spirit must derive partly from the effort which they had had to make to recover from the disasters of the war, but he thought that it was also due to the natural aptitude of the French for planning, and to their incentive bonus schemes.

The track incentive bonus scheme was based on the quality and quantity of the work done as a team in a 3-months' period by each gang and section, and it was calculated from the simple equation $P = i.S.B.t$, where P denoted the monthly bonus, and i the grade coefficient (1 for a platelayer, 1·5 for a ganger, and 2·5 for an Inspector) multiplied by the daily bonus, which in 1950 had been the equivalent of 9*d.* per day. S denoted the section coefficient, which ranged from 0·9 to 1·1, and B the gang coefficient, which ranged from 0·8 to 1·3, and t the number of days in the month which counted for bonus. For each gang $S.B$ was a constant for 3 months. That bonus worked out at about 5 per cent of the monthly basic pay for a platelayer. An administrative bonus scheme had been successfully introduced for the staff working in district offices and head offices.

French railway track was not very inspiring at first sight, because of the cracks in the sleepers and the fact that the rails were only fastened down direct with coach-screws; but the ultimate test of track was the smooth running of trains, and that was certainly very good indeed.

He would like to emphasize Mr Thurtle's remarks about the detailed planning of day-to-day maintenance work, because it was the largest single item in the civil engineer's budget. In any French Inspector's office, the walls were covered with all kinds of work-tables and programmes and progress charts for day-to-day maintenance work, which, of course, was bound up with the incentive bonus scheme. Those charts had been prepared by the Inspector himself. The Inspectors were rather different from the British type and were not promoted gangers.

On p. 399, Mr Thurtle had referred to re-laying an average of about 500 metres (550 yards) in 1 day. On the Nord Region Mr Manson had seen two mechanical ballast-cleaners used instead of one, and the week-day average was 800 yards. On one occasion on the Paris-Brussels main line he had seen 1,200 yards re-laid and re-ballasted in 1 day under single-line possession.

The French system of studying every kind of track work in detail was very interesting, and every job, however small, was carried out to a standard drill.

In the Nord Region there was one central depot for all civil engineering work, at Moulin Neuf, where all new track was prefabricated and all switch and crossing work made, and all serviceable materials were reconditioned. That depot was the largest in Europe. It had twenty gantry cranes and 25 miles of track, and covered 160 acres. There was a Goliath crane in the sleeper yard which had a span of 200 feet. One of its three hoists could pick up six old sleepers on which the ganger in the wagon had already marked the grading, and by means of independent electrical controls the hoist driver could drop each sleeper on to its appropriate stack. In that way the sleepers were unloaded and sorted without any manhandling.

All over French Railways an immense amount of new structural work had been put up to replace what had been destroyed in the war. One example was a new motive-power depot near Lille, which had been under construction in 1950. There were eight wet ashpits in a row, with a gantry crane for removing the ashes which also served a sand drier. The gantry was of reinforced concrete, and in fact most of the structural work in France was of reinforced concrete, because they had little structural steel but plenty of timber for shuttering. Other examples were the standard locomotive houses, of which there were about forty in France, and which showed the quality and the grand scale on which French Railways had been built up since the war.

Mr A. H. Toms expressed his interest in the fact that all the senior staff in the Civil Engineer's department in Holland must be engineering graduates. He was, however, sorry to note that, for certain grades at any rate, promotion was automatic and largely dependent on length of service: had that any effect on initiative? With regard to what was said on p. 382, were Dutch and other Continental Inspectors in later life more efficient at their work, owing to their different training, than the corresponding British Inspectors, for the older generation of whom he had a very high regard? Also in that connexion, did the older men resent working under a young man straight from college?

Had any signs of fracturing been detected in the reinforced-concrete slabs under the track, which were referred to in the last paragraph on p. 385? Mr Toms would be glad to know since that slab had been constructed after discussions between Mr Cuperus and the Chief Civil Engineer of the Southern Region, following an experiment carried out at Ditchling cutting in Sussex.

In *Figs 2*, p. 387, figures were given for the strength of clay, but it was not stated whether those figures were the shear strength of the material or a particular cone resistance, which might be quite different. If they were different, was there a simple factor which could be used to convert one into the other? Recently he had had occasion to make tests with a vibrating compacting machine of the type described in the Note, and to witness tests at the Road Research Laboratory of another type. Experience seemed to indicate that they were very good machines for producing high density and a good vertical density-gradient, and that they compared favourably in those respects with the Frog rammer. He wondered why that machine was not used to compact the full depth of ballast prior to laying the track, whereas he understood from Mr Inglis that the Matisa tamper had been run to and fro over successive lengths.

Had the high initial cost of cast manganese-steel crossings been proved to be more than justified by the reduced maintenance and extended life, as seemed to have been the case in America, where he had seen them in 1950 used extensively in yards and on main lines ?

He was puzzled by a reference in *Figs 11* to a task called "Measuring thickness of sleepers." Was that a routine in relation to the renewal of sleepers ? If so, there was no mention of boring the sleepers to detect whether they were rotten.

It was interesting to note, from p. 397, that the thermit process was still in use for rail welding, whereas he believed that British Railways and the London Transport Executive were of opinion that flash-butt welding was the right method to use. The Americans used either flash-butt or oxy-acetylene pressure welds, and seemed to obtain very good results.

In connexion with the use of long rails, were they anchored ? American success with laying welded rails seemed to be bound up with the extensive anchorage of those rails, particularly when approaching the end of a long section.

In 1946, he had had the privilege of riding in the Mauzin track recording car and its attendant car which was used to record the lateral forces produced on the wheels of the locomotives. He had also studied the record which had been produced by the track recording car and had had an opportunity of going through the theory of its working. Mr Toms felt certain that whereas it recorded a low rail joint or a short depression, with eight-ninths of the true value, on the other hand a long slack of the order of 60 feet or so—which, if occurring alternately left and right, might cause severe rolling of the locomotive—was recorded at only about half its actual value. That seemed to be a fundamental criticism. Whatever track recording machine might be adopted, it should be made very clear to the Inspector over whose length it was running that he was still held responsible for examining the track personally and satisfying himself that what was apparently indicated on the record was what was in fact wrong with the track. The Hallade machine seemed to him to be very simple in its functioning. It had many disadvantages, but after a lengthy experience of its use, and after overhauling such machines and making a small one for demonstrations, he had come to the conclusion that if the records were studied over a long period it was only rarely that the Hallade machine did not show some characteristic to indicate that there was something wrong at a point in the track ; and if the track were inspected something would be found which would account for the indication on the record.

Mr H. T. Horsfield asked Mr Inglis to give some further details of the independent contracting firm controlled by the Netherlands Railways to which he had referred in his Note. Did that organization, in addition to competing with private firms for railway work, also compete with them for non-railway work? If it confined itself solely to railway work, in what way could it be said to have advantages over direct labour under full railway control?

Last summer Mr Horsfield had had an opportunity to see work being done on the Danish State Railways, and a particular point of interest was a flyover bridge by which a track for slow passenger traffic passed over two goods lines at an angle of 12 degrees. Two schemes had been prepared for reinforced-concrete structures. The first had been on more or less conventional lines, with a concrete-slab deck carried on no fewer than forty-four columns. The second was for a rather deeper deck carried on ten columns; of those columns, six were spaced on the centre line of the upper track and the other four were in two pairs, each pair straddling one of the lower tracks. The deck was the normal width for a single track, except at two points where projections were necessary to span the lower track, across the heads of the twin columns. The cost of the second design had been estimated to be 20 per cent less than the cost of the first and more conventional one, and the bridge had now been constructed according to that second design. He thought that the revised design of the bridge was much better to look at than the confused elevation of the first design with forty-four columns would have been.

The main reinforcement for that bridge was a ribbed high-tensile steel manufactured in Denmark, with a working tensile strength of 27,000 lb. per square inch. That, he had been told, was almost always used, and was far more economical than the round mild-steel bars which were commonly employed in Great Britain.

Under the welfare agreements with their staff, the Danish State Railways provided in all their depots, workshops, locomotive depots, and so on, shower-baths for the staff, and had done so for the past 15 years at least. He had been told that wherever circumstances permitted it they were now installing showers in the plate-layers' cabins.

Mr A. G. Ticehurst asked what was the purpose of the interchange of staff visits with other railways? If the object of it was to benefit the railway organizations concerned, he thought that the two Introductory Notes clearly indicated lines of action which could be followed in Great Britain with advantage.

Mr Inglis had drawn attention to the organized training given to young artisans and students on the Netherlands Railways. That degree of training, so far as Mr Ticehurst was aware, was not given on British Railways, although they had several establishments which catered for refresher courses for more mature railway servants, both technical and clerical. That was one of the matters in which British Railways could very well follow the lead of both the Netherlands and the French Railways, by instituting some practical and widely-applied system of training for the young men that it was hoped to recruit to the railway service. It was well known that there was at present a dearth of apprentices in all the skilled trades. If such a scheme were in operation it would make the railway service appear much more attractive to young men than if they had to enter it and simply take "pot luck."

Mr Thurtle's instructive remarks on day-to-day maintenance programmes in France indicated that division of labour simplified supervision and enabled the expected output to be attained. If adopted in Great Britain, it should also make possible the adoption of job costing. That was, perhaps, a source of contention and not an easily accepted principle, but it was of major importance, in Mr Ticehurst's view, especially now that so much attention had to be given to economic working. He viewed job costing as the lifeblood of an industrial undertaking, and the adoption of the very simple methods which Mr Thurtle had indicated would make it possible to see just where the money went and to control it. Mr Ticehurst believed, moreover, that an incentive scheme was being considered for track maintenance work, and the plan to which he referred could form the basis on which such a scheme could be started.

Something had been said about welfare arrangements, and he heartily agreed with the need for those.

The progress of mechanization was in its infancy in Great Britain as in other countries, but it was difficult adequately to appreciate what was going on merely by reading publications and articles on the subject in the Press. It might be more valuable for a regular interchange of staff visits to take place and for a tribunal to make regular visits to other railway organizations, with the express purpose of bringing back ideas which could be carried out in Great Britain. Mr Smith had referred to a dumper scheme with single-line working with a ballast-cleaner, and something on those lines might be done right away. The ballast-plough shown by Mr Inglis was another example of a direction in which something should be done. Some years ago Mr Quartermaine had instructed Mr Ticehurst to produce one, and after some delay he had got a prototype fixed up on the side of a well wagon, and that had been tried and had seemed to work satisfactorily, so that Mr Quartermaine had urged its adoption. But that had been prevented by Mr Ticehurst's inability to get any more wagons to carry ploughs. No money was allowed, so far as he was aware, to expend on experiments and developments of that kind. If a few thousand pounds a year were allocated for the purpose, ballast-ploughs and all manner of other things could be developed.

All the points noted by Mr Thurtle under the heading of "General Impressions" were well worth noting, and especially the administrative posts held by qualified engineers. That was perhaps the secret of the success of the French Railways.

Mr M. B. Macrae referred to the pre-stressed concrete masts used by the Netherlands Railways for overhead electrical equipment, which had not yet, he observed, been tried out in Great Britain, where the only overhead equipment in use, or about to be brought into use, employed rolled steel joists. However, provisional designs had been got out for pre-stressed concrete masts. The main difference between the design developed in Great Britain, by the Eastern Region of British Railways and that of the Netherlands Railways was that the former was an H-section mast, whereas on the Netherlands Railways a lattice type had been used. Which had the more pleasing appearance was a matter of opinion, but it seemed to him that the cost of the Dutch type might well be more than that of the H-type which the Eastern Region had in mind. The lattice design, moreover, was perhaps not quite so good where reversals of stress might be looked for in such a structure.

On the Eastern Region no overhead masts carrying electrical equipment had yet been erected, but an opportunity had presented itself to try out a prototype on the Manchester-Sheffield electrification, where the Ministry of Transport required that there should be warning load-gauges at public level-crossings to ensure that road loads of greater than normal height did not foul the overhead wires. The Eastern Region had produced posts and beams which could be used in substantially the same way when overhead equipment had to be provided. There were bells fitted, which would ring as a warning to the driver when anything hit the structure. The posts were 21 feet long and weighed about 12 cwt., and the overhead beam was about half that weight. It must be understood, however, that those weights would be increased somewhat when it came to the actual design carrying the loads required for overhead equipment. The beams and posts were made on the Hoyer system, with some eight wires in the case of the posts. It had provided some valuable experience in showing how that type of structure behaved.

Finally, had any difficulty been found in the matter of the foundations for those overhead structures in Holland ? The general type of countryside in Holland might be different from that in Great Britain, where great difficulty had been experienced, when digging the foundations, in getting rid of the excavated material, which was apt to be spread amongst the ballast and did not do it any good. Were the Dutch any more successful in getting rid of it ?

The Chairman observed that the organization of the nationalized French Railways was rather different from that of British Railways. A good deal was heard in Great Britain about the dangers of centralization. He thought he was right in saying that the S.N.C.F. had many hundreds of engineers and draughtsmen at the headquarters of what corresponded to the Railway Executive in Paris. Had Mr Thurtle, when attached to a Region of the French Railways, heard any comments, adverse or otherwise, on that subject, bearing in mind that the British Railway Executive had no Civil or S. & T. engineering drawing offices at Headquarters, and relied for that on the Regions ?

Mr R. A. H. Burton said that Mr Inglis, when dealing with soil mechanics, had mentioned in connexion with investigational methods the use of cone testing and described a method which did not appear to be greatly used in Great Britain. It had been reported to be extensively used in Sweden, and certainly 10 years ago the Americans were using it in the form of the North Dakota cone test. It would obviously seem to provide, at first glance, a quick and simple method of testing a clay shear strength, because large numbers of tests could be made fairly quickly and the procedure was similar to that employed when using a Proctor needle in a compaction test. If a satisfactory correlation with the actual shear strength of the clay could be obtained, it seemed to be a valuable method to develop in Great Britain. He had heard it said that a satisfactory correlation had not been obtained in experience in Great Britain, but, since the Dutch seemed to rely on the test extensively, it would be interesting to know whether they had obtained satisfactory correlation and were pleased with it, or whether, perhaps, they did not correlate it with the actual clay shear strength but simply found that it could be used as a satisfactory basis of comparison.

Mr E. C. Cookson referred to the manganese-steel crossings mentioned by Mr Thurtle, with the steel strut from the crossing-nose to the guard rail, and said he believed that it was the practice on the French Railways in a very large number of cases to use a ramped throat in the crossing, so that the flange of the wheel relieved the crossing-nose of some impact. He also believed that the French worked to much flatter crossing angles than was done in Great Britain and depended on the distance between their crossing guard-rail and the nose of the crossing to keep themselves on the right side of the flat crossing.

What he intended to say next was not intended to detract in any way from the merit of the two Introductory Notes which had been presented, but he agreed with Mr Ticehurst that it was desirable to have some regular body to follow continually the developments which were made in other countries. For instance, Mr Thurtle had mentioned the diplory method of re-laying, and the portal frames which were put on rails by the side of the track. Mr Cookson understood that the French had now developed a method of using portal frames which did not require rails by the side of the track.

Mr A. K. Terris observed that the Chairman had asked for a report on the use of the diplory equipment on the Eastern Region, which was, of course, doing the work on behalf of British Railways as a whole. The equipment concerned, although on the same principle, was not the same as that used by the French Railways, but was something which the Region was trying to develop. Some experience of it had been gained in Norfolk, where it was thought that the conditions and the fact that there were many single-line branches would justify its use.

There had been several teething troubles. On the first two occasions it had been found, when the accountant had a tally of the cost, that the cost of the re-laying was considerably more than hand re-laying. Mr Terris did not say that there was anything wrong with the method; it had been the design which was at fault, because some of the diplory wheels had seized and so had the winches which lifted the track on to the formation. Nevertheless, there was some evidence of the value of the French method.

In thinking of that system, it was necessary to start by comparing the British Railways with the French Railways. So far as track possession was concerned, how did this country compare with France? Those who had been to France all said that the conditions were altogether different. Those who were concerned in Great Britain with re-laying operations from week to week, and who tried to see how mechanical equipment could be developed in re-laying operations, were always faced with the problem of possession. So far as diplory equipment was concerned, the single lines—and each Region had a great mileage of them—were the places where such equipment, when it was developed, would be of value. The Eastern Region was persevering with the diplory equipment, and hoped in time to improve on first experiences.

There were several other points which stood out when trying to compare British and Continental methods, particularly the top-level organization of the Engineering Department of the French Railways, and of the Netherlands Railways also. It was obvious from the figures which had been quoted that they had a top-level organization which did not exist in Great Britain. Considering that there was a staff of five hundred in the head office of a section of the French Railways, compared to that of a comparable Region of British Railways the difference was tremendous. In the Districts that difference still continued. If the assessment of work to be done and the dissection of day-to-day maintenance were necessary to provide the standard obtained on the French Railways, was the additional cost reflected by a comparable difference in the work standards of the two countries? He very much doubted it.

In spite of that he was of opinion that, as he had stated previously[1], it was necessary in Great Britain to face the need for change, and there would be general agreement that circumstances outside made it appear that the time had come.

As he saw it, it was a question of determining what quantity of supervision was required outside and whether there should not be technical assistants superimposed over the inspectors to deal with modern conditions.

---

[1] A. K. Terris, "Economy by Supervision." Proc. Instn Civ. Engrs, Part II, vol. 1, p. 114 (Feb. 1952).

Mr A. H. Cantrell said that Mr Thurtle had mentioned that the French Railways did their maintenance by two separate methods, one being a complete repair and the other a partial repair. Was there any definite period of time between the complete repairs, or did it depend upon the condition of the material? From Mr Thurtle's Note it would seem that there was a time basis.

In those repairs it appeared that the markings on the track alignment monuments were altered at each complete repair. He did not know whether that was in fact the case, but it seemed to him that if suitable monuments existed those monuments should stay there, and that realignment should not be necessary each time the complete repair was carried out.

Great use could be made of void-meters, but Mr Cantrell found it very difficult to get the men to use them. The chief objection was the time and labour spent in driving the spikes of the type usually employed in Great Britain. The French type seemed rather cumbersome, although evidently it could simply be pushed down into the ballast, but it seemed to him, however, that the void had still to be measured separately, since it was not indicated by a dial, as with the type used in this country. At the moment, on the Southern Region, a new type was being tried for which no spikes had to be driven, nor was it necessary to take out any ballast; it was just put on the top of the ballast under the rail. If that type could be developed satisfactorily it might provide the answer which was needed for measured shovel packing.

Mention had been made of the fact that French construction was mainly in reinforced concrete. In 1946, Mr Cantrell had had the honour of going to France to see how the French Railways were getting on with their bridge reconstruction after the war, and he had found that most of the work was in reinforced concrete. The engineer who had taken him round had explained that the chief consideration was coal consumption. Steel meant coal, and for each structure the quantity of coal required for steel and reinforced concrete was calculated. For any sizeable structure reinforced concrete showed a big saving in coal and was therefore adopted.

Mr J. G. F. Inglis, in reply, first pointed out that all the information he had to pass on was second-hand and from memory.

Mr Smith had asked for further information about the bulldozer scoop described on p. 389 in the Introductory Note. So far as Mr Inglis could remember, the machine seen in the Netherlands was manufactured in the United States. He had made enquiries at the Building Exhibition this year at Olympia, but the only comparable British machine was mounted on a standard agricultural tractor, and was not available on a bulldozer or other small track-laying vehicle. The British machine was also somewhat larger. This type of machine had a number of advantages for small jobs, where it showed a great saving over hand methods, but was not so economical for larger jobs. The machine in use in the Netherlands would only load on to flat wagons when standing at sleeper level or just below.

Mr Toms had raised a number of points. The system of automatic promotion was nothing unusual and was practised in the British Civil Service and the Armed Forces, up to certain grades or ranks. It seemed to be a good thing in a large organization to have promotion on an automatic basis in the lower grades; otherwise it was often a matter of chance. Whether the Dutch Inspectors were more efficient in later life than the British was a very difficult question and Mr Inglis was not prepared to give an answer. He had had no experience of Dutch Inspectors.

Mr Toms had also asked whether any fracturing had been detected of the continuous reinforced-concrete slab on the Gouda–Oudewater line. Mr Inglis had not heard of any but did not know what inspections had been made.

Reference had been made by Mr Toms and Mr Burton to the cone resistance test. The figures quoted in the Note referred to the direct pressure required to force down the standard cone of 10 square centimetres area with a cone angle of 120 degrees. So far as he was aware, no relationship had been established between that and shear strength. The test was used qualitatively although in a quantitative way; it was used to measure relative rather than absolute values. When calculations had to be made, some variation of the triaxial test was employed.

Mr Toms had also asked a question about the consolidation of the ballast in two of the blanketing jobs described in the Note, and wanted to know why it had been carried out with the Matisa tamper and not with the vibrating plate machine. The main reason was that it was quicker to get the ballast back if it were tipped from hopper wagons travelling over the track being ballasted, in which case, of course, only the Matisa could be used. Moreover, Mr Inglis thought that if the ballast were to be put down hard with the vibrating plate machine, trouble with a centre-bound road might be experienced later on. If it were left fairly loose and then drawn down and packed tight where most required, that was, under each individual chair or baseplate, there would result in time—it might take longer—a well-consolidated track.

In commenting on Mr Thurtle's Note, Mr Toms had referred to the use of the thermit welding process for joining rails. In the Netherlands, welded rail-joints, of which there was a considerable number since the new heavy-section rail was being laid in 120-foot lengths (the Dutch answer to the "joint" problem being to reduce the number of joints) were all being carried out by the flash-butt process. Figures had been given for thermit welding as used previously and the percentage of failures was very much higher than with flash-butt welding.

Mr Horsfield had asked about the contracting firm, referred to on p. 382, which was controlled by the Netherlands Railways. Anticipating that that question might be asked, Mr Inglis had obtained some particulars. The organization had been founded in 1921 by the Netherlands Railways as a limited liability company. It had undertaken in its earlier days some non-railway jobs, but was now used almost entirely for purely railway work, although it did a little work for various subsidiary companies of the Netherlands Railways. It tendered for all sizes of contract let by the Civil Engineer's Department and undertook all types of work except bridge-building in steel, work being undertaken sometimes in co-operation with private firms. The work undertaken in 1950 was valued at £1,200,000, or almost 50 per cent by value of all the contracts let by the Railways, of which 78 per cent had been obtained by competitive tender.

Mr Ticehurst had asked what was the reason for the interchange of staff. Mr Inglis and others concerned had been puzzled by that too, and they had never found the answer to it. Mr Ticehurst had also asked about the training of artisans, and Mr Inglis pointed out that that was a national system, not a Railway one. There was in the Netherlands a far greater education- and examination-consciousness than existed as yet in Great Britain, although the British people might in time come to approach it. He had found also in the Netherlands that a great interest was taken in not only their own railways but the railways of other countries. The foregoing referred primarily to the engineers, who were, as pointed out in the Note, a much smaller and more homogeneous body than in Great Britain, and were able to hold regular meetings and also to circulate a selection of railway and other technical publications in several languages. In the Netherlands, many people, including the station staff at large towns, spoke four languages.

Mr Macrae had asked about the pre-stressed concrete masts used for overhead electrification. So far as the foundations were concerned, in the Netherlands very considerable lengths of track were on shallow embankments (see p. 390) and it was possible easily to distribute along these embankments any excess soil remaining after re-filling and ramming round the pre-cast bases. An interesting feature was that, the mast having been erected and centred, the annular space between it and the base-block was filled with stone chippings instead of being grouted up. This form of packing maintained the mast rigidly in position, and if withdrawal should be necessary, it would be greatly facilitated.

Mr Burton had asked about the cone test, and Mr Inglis had already tried to deal with that. He had found no use of the vane test apparatus in the Netherlands, which he believed was in use in Sweden. The Dutch were not carrying out so much research on soil mechanics as was being done in Great Britain, at any rate so far as the respective Railways were concerned, but they had developed their methods and were applying them. They had, he believed ten sets of cone-test apparatus available, and they had a vast amount of information collected and recorded, which could be produced at a moment's notice if required.

Mr Terris had referred to re-laying operations. In the Netherlands, with long rails, the trend was away from mechanization; they had not found a way of satisfactorily re-laying by mechanical methods when using those long rails.

Mr R. G. Thurtle remarked that he had been most interested in the method described by Mr Smith for dealing with ballast cleaning with the occupation of one track only. He did not think that that had been developed when he was in France, or at any rate he had not been able to see it. He would couple with this matter the remarks made by Mr Ticehurst about the money available in Great Britain for development; it would not be difficult for young engineers in Great Britain to originate such ideas, if they could have a little money to play with.

Mr Ticehurst had also mentioned the advantages of the French system of organizing work and detailing methods of work, to which reference had been made in Mr Thurtle's Introductory Note but Mr Thurtle had also tried to point out that close supervision would be required to get the best out of that system—closer supervision than seemed to be given in Great Britain at the moment. The type of men available in future as Inspectors would have some bearing on that, and labour being recruited at present seemed to Mr Thurtle unlikely to contain men of suitable calibre to carry the responsibilities imposed on Permanent Way Inspectors by the present organization in Great Britain.

Mr Toms had raised a whole series of questions. Mr Thurtle did not feel able to answer the question as to whether or not manganese-steel crossings were justified from the cost aspect, but he knew that the French expressed nothing but satisfaction with them, and he thought that towards the end of their life—which, of course, could be extended many times with electric welding—they must be a much more solid job than crossings in Great Britain, which at the end of their life were almost rattling to pieces.

With regard to the object of measuring the sleepers, the sleepers used by the French were not squared, as were those in Great Britain, so that each sleeper was likely to have a slightly different depth. To avoid having hard places or slacks when replacing defective sleepers it was necessary to measure both the old and the new sleeper and make adjustments accordingly when putting in the replacement.

The use of thermit welding for joints was confined in France to depots and branch lines. It had, of course, some advantages; it was a mobile process and was applicable to eliminating fishplated joints in depots, whereas flash-butt welding would not be suitable for that type of work. With regard to anchoring long rails, his remarks on thermit welding did not apply to long rails in running lines, but he did see one such experimental length which the French had laid with flash-butt-welded rails carried on longitudinal concrete blocks over a length of about ¼ mile, with expansion switches at each end. These rails were scribed throughout their length with monuments set up beside them, so that the movement of the rail could be seen. When he was there, in the height of the summer it was found that, whilst there was a certain amount of movement towards the end of the lengths, where the expansion switches allowed it, for about two-thirds of the length, in the centre, there had been no movement whatever, which suggested that the rails were in compression and retained by the heavy concrete blocks.

With regard to the Mauzin coach, Mr Toms was at an advantage, because Mr Thurtle had never been able to ride on it; but it seemed to him that there might still be a place for the Hallade recorder, which measured passenger comfort, as it were, as against track defects, especially if it was regularly used in the same vehicle, as on the Western Region. It seemed to him that possibly the Mauzin coach, which provided a very accurate record of conditions, might show up defects in the track which did not affect its safety, and which might not affect the comfort of passengers in the vehicles running over it. If that was not watched, attention might be devoted to putting right defects which did not warrant it.

The Chairman had asked whether he had heard any adverse comments on the French equivalent of the Railway Executive. It might make the Chairman envious, but he had not. Possibly that was due to the traditional politeness of the French.

Mr Cookson had mentioned the manganese-steel crossings and the strut used in conjunction with them. The ramped throat was used in France, and the cast crossing lent itself to that. The French also used much flatter angles than were employed in Great Britain, but Mr Thurtle had formed the opinion, since the strut was standard on crossings of all angles, that the basic reason for it was the tendency for their timbers to shrink.

Mr Terris had mentioned that single lines seemed a useful field in which to use the diplory and portal method of track re-laying. Mr Thurtle agreed, since any occupation of those tracks must be a total occupation, and that seemed to be the only means of applying prefabrication methods to such lines; but even if traffic conditions in France were different from those on our main routes, there was surely a very large route-mileage of double track in Great Britain with traffic conditions similar to those in France, and it was on such routes that the maximum benefit could be obtained from that method of prefabricated re-laying with single-line possession.

Mr Cantrell had referred to the question of complete repair and partial repair. That was done on a time basis. Each District was required to do a complete repair of a quarter of its tracks each year, and a partial repair of a further quarter each year, so that the whole cycle was gone through in 4 years.

With regard to the monument markings, Mr Thurtle did not think that the curves were necessarily re-aligned at every complete repair, but the type of monument used made it easy to re-align curves, supposing one or two monuments, or even a whole length of them, were in time disturbed through the action of the ballast. Over a long period of time they might very well prove better than monuments with lead inserts or fishplates with sawcuts in them—since the last-named methods were more rigid—and once out of position were less easy to modify.

The discussion had seemed to suggest that there was something to be gained from an examination of the methods of engineering departments of other railways. On his visit to France he had certainly been impressed by the close supervision of work there, as well as by the keenness of the French railwaymen. He believed that those impressions had been shared by Mr Manson, who had visited the French Railways in similar circumstances a year later. The reasons for the results achieved were more difficult to be sure of, but the fact that the highest administrative posts on the French Railways were held by qualified engineers, together with the bonus schemes and the great interest taken in staff matters generally, must be among the underlying causes. The fact that their organization gave early experience in handling men was, he thought, another cause.

Mr Inglis's very interesting Note made it clear that the Netherlands Railways had in common with the French Railways a three-tier organization supervising the work on the engineering district. Although neither system might be applicable in Great Britain without modification, it seemed to Mr Thurtle (as Mr Terris had also suggested) that the time might be appropriate to review the organization of engineering maintenance on British Railways, which he believed had not been radically changed since the railways were constructed. In any case he hoped that the points brought to light in the discussion that evening had been of interest.

The foregoing Discussion is not open to Correspondence.—Sec. I.C.E.

AIRPORT ENGINEERING DIVISION MEETING

26 February, 1952.

Sir Hubert Walker, C.B.E., M.I.C.E., Chairman of the Division, in the Chair.

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Airport Paper No. 19

"The Influence of Multiple-Wheel Undercarriages on the Design and Evaluation of Airfield Pavements" *

by

Guthrie Stewart Cooper, B.Sc., A.M.I.C.E.

SYNOPSIS

The Paper opens with a brief history of airfield pavement construction and design methods, and shows how the multiple-wheel undercarriage was introduced in an effort to reduce the loading effect on pavements of very heavy aircraft. The expression "Equivalent Single Wheel Load" is defined. Failure of pavements, both flexible and rigid, is discussed and the effect of interaction of stresses caused by multiple wheels described. Methods are then given for the computation of Equivalent Single Wheel Loads on flexible and rigid pavements, and the application of the Equivalent Single Wheel Load in pavement design and evaluation is discussed.

Graphs are presented for the derivation of Equivalent Single Wheel Loads on different thicknesses of rigid pavement, and also graphs showing the variation of Equivalent Single Wheel Load with pavement thickness, on flexible and on rigid pavements, for several typical aircraft.

The variations which may affect the problem are outlined, and reference is made to other methods which are in use. Mention is made of the particular difficulties arising which affect the aircraft designer, and photographs of some typical multiple-wheel undercarriages are included. Finally, attention is again drawn to the many unpredictable factors, and the limitations of the methods presented in the Paper are stressed.

Introduction

In considering the multiple-wheel undercarriage, it is interesting to examine the factors and developments which led to its adoption. These are related to the history of airfield design, which in turn was dictated by the trend of aircraft laden weights. Broadly, the history of airfield design, up to the present, falls into three phases.

The first phase extends from the early days of aviation to the commencement of the Second World War, a period of approximately 30 years. During the early part of this period aircraft took their conventional shape,

---

* Crown Copyright reserved.

which has persisted to this day. The landing-wheel arrangement consisted of two main single wheels placed transversely in line, which carried the bulk of the aircraft load, and a nose or tail wheel. Wheel loads in this phase generally did not exceed about 15,000 lb. and tire pressures were less than 40 lb. per square inch, although, before the introduction of the "balloon" tire in the middle of the phase, comparatively high tire pressures were not unknown in relatively narrow tire sections, and some of the heavier aircraft carried two wheels on each undercarriage. Such loads and pressures could be carried satisfactorily on grass airfields provided the number of aircraft movements was relatively low, and provided the turf, soil, and subsoil had reasonably high bearing values not unduly affected by rainfall.

Towards the end of the first phase the tendency to higher speeds and smoother aircraft lines began to show itself. The wheels caused considerable drag, and efforts were made to reduce this by using smaller wheels, or to eliminate it entirely by retracting the undercarriage. As the necessity for cleaner lines restricted the space available for stowing the retracted wheels, retracting undercarriages themselves called for a reduction in wheel size. This reduction in wheel size, coupled with increasing wheel loads, led to increases in tire pressures, and it was found that only very few natural grass airfields had bearing values sufficient to carry the heavier higher-performance aircraft at high operational intensity.

This led to the second phase, in which paved runways were provided on all airfields intended to carry other than the lightest aircraft. Little was known at the time regarding the strength characteristics of flexible and rigid pavements, or the bearing values of the soil on which they were constructed. Pavements were not designed from a theoretical viewpoint and airfield engineers relied largely on the experience of the road engineer. Flexible pavements seldom exceeded 12 inches in depth, and concrete slabs 6 to 8 inches thick, on a hardcore base course up to 6 inches thick, were commonly adopted in rigid pavements on weak clay subgrades. Road engineering had proved such pavements to be capable of carrying considerable vehicular wheel loads, sometimes on solid tyres, and it was expected that they would carry aircraft wheel loads of 20,000 lb. at 40 lb. per square inch with the reduced intensity and increased distribution of traffic expected on airfields. These pavements did in fact prove adequate until the advent of the four-engined bomber in 1942, with wheel loads up to 30,000 lb. and tire pressures of 45 lb. per square inch and higher. In 1943, pavements subjected to use by these aircraft began to show signs of failure, and this failure was checked only by extensive surfacing programmes. These results led to greater interest in rational design methods, and the latter were in general use for rigid pavements well before the end of World War II, the first application on military airfields in the United Kingdom being in 1942.

The introduction of rational design marked the commencement of the third phase. Preference was shown at the time for concrete, and almost all military airfields under construction in the British Isles had rigid pavements. This was mainly a matter of expediency, since contractors were equipped with the necessary plant for large-scale concrete construction, whereas, although tarmacadam and asphalt spreaders and finishers were appearing in increasing numbers, they were all required for the nation-wide programme of surfacing the earlier airfield pavements. Bitumen and high-grade tars were also scarce, and, in addition, there was a shortage of the earth-moving and heavy compaction plant required for proper base-course construction for flexible pavements. The preference for concrete led to many concrete pavements being laid on gravel and sand subgrades of high bearing value, on which flexible pavements would have been more appropriate; little work was done on the development of flexible pavement design methods.

The rigid pavement design method used by the Air Ministry for the third phase was based on the theories introduced by the late H. M. Westergaard.[1]

Whilst flexible pavements were, in general, largely neglected in the United Kingdom, an enormous amount of work on the development of flexible pavement design was being carried out in the United States of America. The Corps of Engineers, U.S. Army, did considerable research into various empirical design methods in use by different highway authorities, and performed accelerated traffic tests on a vast scale. Following these investigations, the Corps of Engineers finally adopted the California Bearing Ratio system.[2] This system, originally introduced for road design, was extended to cater for aircraft wheel loads, and design curves for flexible pavements on subgrades of various C.B.R. values were developed. These curves as originally produced made no reference to tire pressure, although it is believed that 60 lb. per square inch was the figure intended. The Air Ministry adopted the Corps of Engineers C.B.R. method for flexible pavement design with one detail amendment, namely, the C.B.R. value of the subgrade adopted for design purposes was not the soaked value as stipulated by the Corps of Engineers for certain cases. The probable maximum moisture content of the subgrade under stable conditions after construction of the pavement was estimated, and the C.B.R. value was determined at that moisture content and at the density likely to be achieved in construction. More recently a system making use of plate bearing tests has been developed by the Air Ministry. The view is taken that a bearing plate, having a contact area comparable with the area which will be loaded when the pavement is in use, gives a more representative figure than the C.B.R. test by eliminating local variations, and by applying stress simultaneously to all the various soil strata which might occur beneath subgrade level. Pavement thicknesses calculated from plate bearing tests and from field C.B.R. values have been found to agree fairly closely.

---

[1] The references are given on p. 446.

While rational design methods were being introduced for airfield pavements, giant aircraft were also being planned. The B-36 was under construction in the United States and the Bristol Brabazon in Great Britain. Both of these aircraft had all-up weights in the region of 300,000 lb., and it was apparent that very few airfields in existence could carry them if orthodox single wheels were mounted. To avoid an enormous programme of building new airfields and strengthening existing pavements for the operation of these aircraft, discussions took place in both countries between the aircraft designers and the airfield pavement designers, with the result that both aircraft were designed to have multiple-wheel undercarriages. One B-36 was, however, built with single wheels. The multiple-wheel undercarriage decreased the effect of the aircraft loading, and consequently reduced the load-carrying capacity required in the pavements on which it might be used. It was also of assistance to the aircraft designer, since the weight of the assembly and the storage space required were, in some cases, less than those for a single-wheel assembly. There are one or two instances before World War II of aircraft being designed with multiple-wheel undercarriages primarily for this purpose.

The Equivalent Single Wheel Load

The fact that the multiple-wheel undercarriage would reduce the effect of the load on the pavement was apparent, but the amount by which it would do so was not so apparent, nor was it quite clear how this amount might be calculated. It was realized that there would be some degree of overlap of the stresses caused by the various wheels, which would result in the total effect of all the wheels of the undercarriage being greater than the effect of each individual wheel, and the airfield pavement engineer had to find a method of assessing the extent of the effect of this overlap.

To simplify the problem, attempts were made to express the total loads of the multiple-wheel assemblies in terms of an Equivalent Single Wheel Load. This term has been defined as follows :—

> " The Equivalent Single Wheel Load for a particular group of two or more relatively closely-spaced wheels is the isolated single wheel load which, operating at a specified tire pressure, produces critical effects in a particular airfield surface, base, and subgrade equivalent to those produced by the group of wheels."

When the Equivalent Single Wheel Load of a particular aircraft and the associated tire pressure (normally the maximum tire pressure of the wheels of the multiple assembly) are known, it is possible to calculate the Load Classification Number [3] for the aircraft and decide whether it can land safely on the airfield under consideration.

The problem is, therefore, one of investigating the equivalent single wheel load and determining methods of evaluating it under different sets of conditions. With the exception of a few airfields where other methods of construction have been employed, all airfield pavements are either flexible or rigid, and these two main types are discussed in this Paper.

FLEXIBLE PAVEMENTS

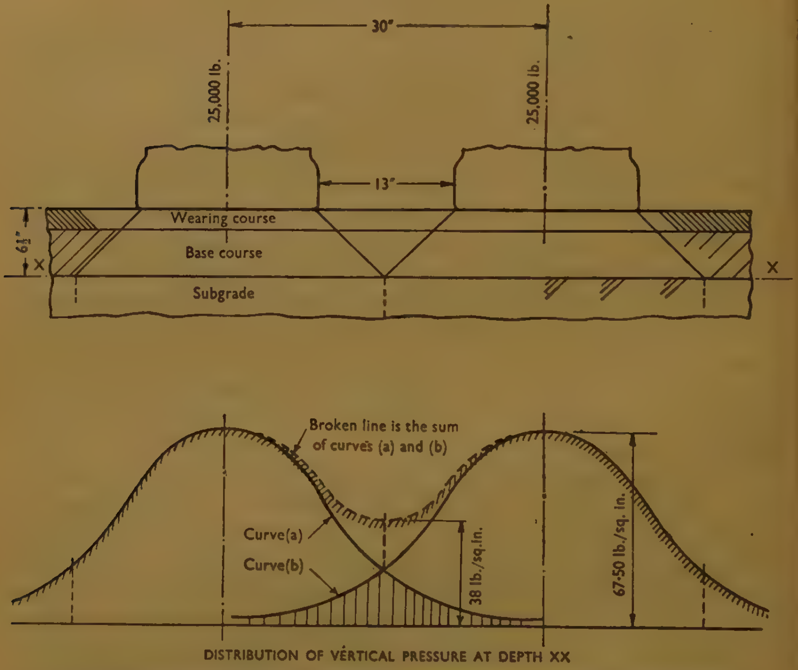
The equivalent single wheel load for a particular multiple-wheel undercarriage is that which will produce critical effects equivalent to those produced by the multiple-wheel undercarriage. The ultimate critical effect in a flexible pavement is failure, either by cracking on the surface, rutting, or extensive heave. Cracking on the surface, when caused by loading, is normally due to shear failure of the surfacing itself. Rutting is normally due to failure in the top layers of the base course, resulting in punching shear failure of the wearing courses through lack of support from the base. Minor rutting may also occur through subsidence on consolidation under wheel traffic, owing to incomplete compaction of the base and subgrade during construction. Heave of limited extent may be associated with rutting, but extensive heave is almost always caused by failure of the subgrade itself, resulting in upward displacement of the subgrade and base-course material on either side of the area on which the applied load exerts vertical pressure.

The different types of failure which may occur introduce special difficulties. For example, where the critical loading on a certain multiple-wheel undercarriage might cause failure in the subgrade of a particular pavement, a single wheel at the same contact pressure as the wheels of the multiple-wheel undercarriage might produce failure in one of the upper layers of the base at a lower load than the load at which it would cause subgrade failure. Or it might cause failure in the wearing courses at a lower load than that at which it would cause subgrade failure. Factors such as these complicate the problem of the equivalent single wheel load, since it is apparent that the latter must be the lowest load which produces failure of any kind, so far as the particular multiple-wheel undercarriage and pavement under consideration are concerned.

Certain assumptions have to be made if the complications mentioned are to be overcome. In the method put forward in this Paper for computing equivalent single wheel loads for flexible pavements, it is assumed that the construction of the pavement is such that, under any combination of single or multiple wheels, and within a reasonable range of contact pressures, failure occurs in the subgrade before it occurs at any other level. This assumption will normally hold in a carefully designed pavement where a margin of excess strength is included in each of the base-course layers.

With the above assumption, the problem becomes one of comparing the stressing conditions caused in the subgrade by the single- and multiple-wheel assemblies. *Figs 1* and *2* show the vertical stresses caused by a twin-wheel assembly on a relatively thin flexible pavement and also on a deep pavement, and the considerable increase in the degree of interaction of the effect of the individual wheels with the deeper pavement will be noted. In all cases the contact area is assumed to be circular.

Figs 1



TWIN WHEELS ON THIN FLEXIBLE PAVEMENT. VERTICAL PRESSURE ON SUBGRADE

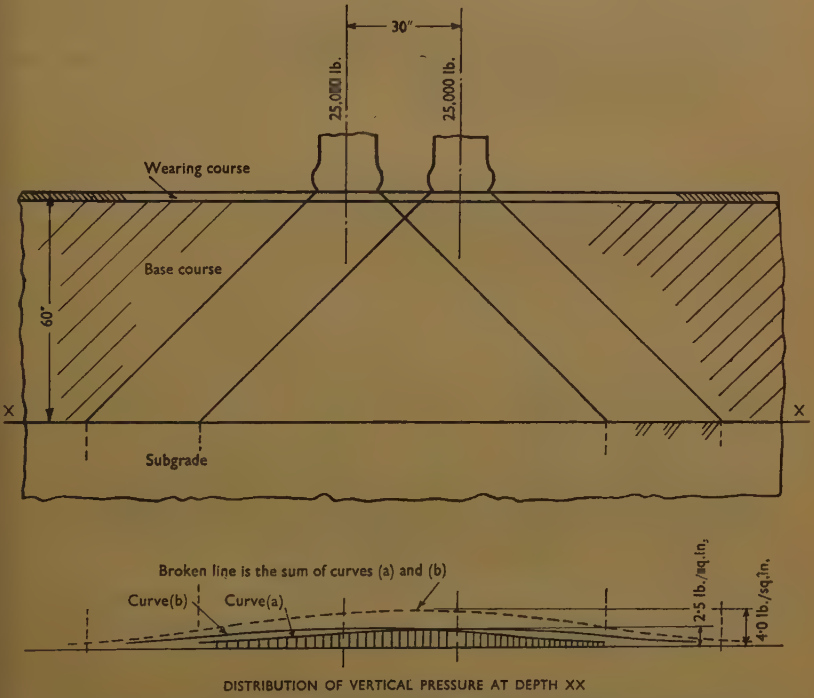
The vertical pressures have been computed from a modification of the Boussinesq equation,[4] as developed by Fröhlich.[5] The normal Boussinesq formula is:

$$q_z = q_s(1 - \cos^3 \alpha) \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where q_z denotes the vertical pressure on the axis of a load at depth z (lb. per square inch),
q_s ,, ,, intensity of applied load (lb. per square inch),
and α ,, ,, angle subtended between the axis at depth z and the edge of the loaded area.

The symbols are illustrated in *Fig. 3*. The formula assumes that the mass under the load is of uniform strength throughout, to infinite depth. In practice, in a flexible pavement, the strength (as represented by the modulus of elasticity) decreases with depth and it is this consideration which indicates that a more accurate assessment of the vertical stress is given if the concentration factor of 3 (the power to which cos α is raised) in the Boussinesq formula is reduced to 2. The validity of this theory is confirmed to some extent by the investigations carried out by O. J. Porter[6] on behalf of the United States Corps of Engineers at Stockton Airfield.

Figs 2



TWIN WHEELS ON THICK FLEXIBLE PAVEMENT. VERTICAL PRESSURE ON SUBGRADE

From the above it is possible to compute the maximum vertical pressure occurring on the subgrade under a particular multiple-wheel undercarriage. This maximum stress normally occurs under one of the wheels in a relatively thin pavement but, in a thick pavement, it usually occurs under the centroid of the group of wheels, on account of the degree of interaction between the stresses caused by the individual wheels. It is also possible to determine the wheel load which, at the same constant pressure as the group of wheels, produces the same maximum pressure in the subgrade. This wheel load could be regarded as the equivalent single wheel load.

It is thought, however, that the equivalent single wheel load determined by the comparison of absolute maximum vertical stresses in the subgrade may not be truly representative. The maximum stress occurs at a point, and a slight local overstressing may not result in a general failure. Accordingly, a "search plate" method has been developed. In this method consideration is given to loading on an area of 30 inches diameter, representing the "search plate," laid on the subgrade. By trial and error, with the plate in various positions on the subgrade, the maximum load carried on it is determined, and, from that, the maximum average vertical pressure on the subgrade. The equivalent single wheel load is, then, the load which, at the same contact pressure, produces the same maximum total load on the search plate on the subgrade as the multiple-wheel undercarriage. In the case of the single wheel the centre of the search plate is on the axis of the load.

Fig. 3

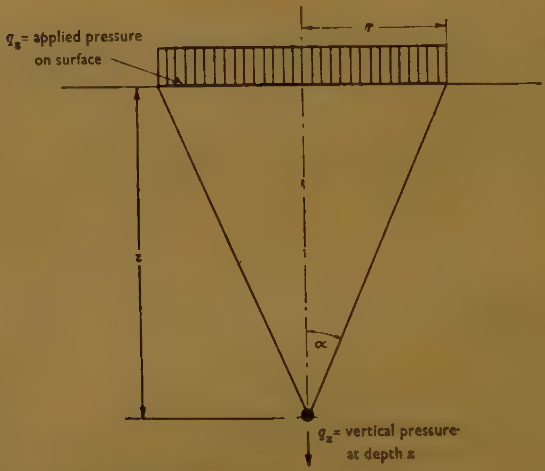
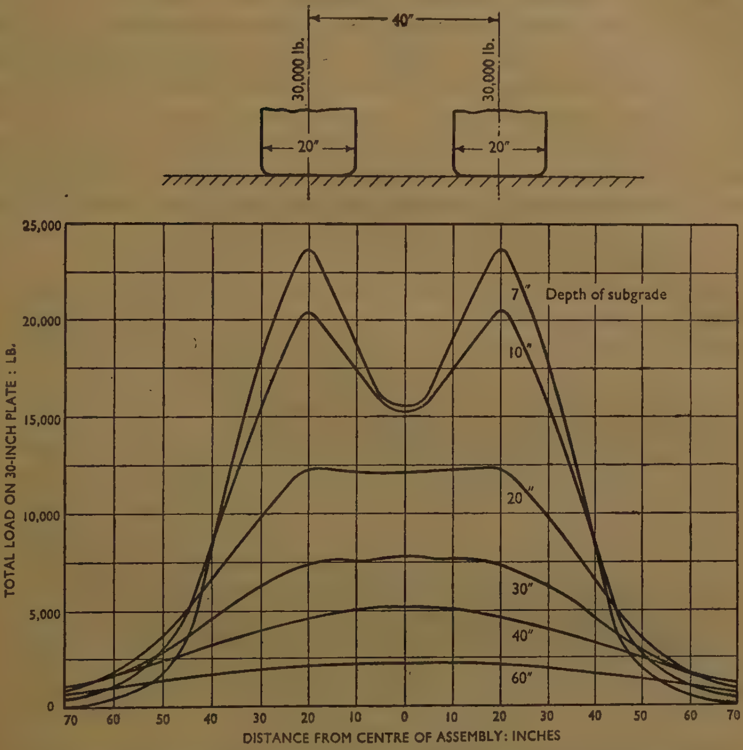

q_s = applied pressure on surface
r
z
α
q_z = vertical pressure at depth z

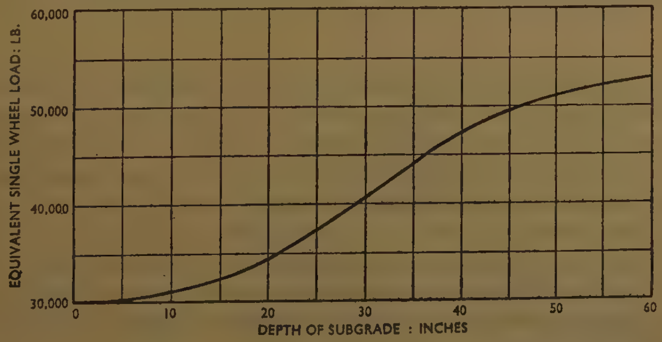
Diagram Illustrating Boussinesq Formula

The 30-inch plate-diameter was selected because this is generally accepted as the size above which, for all practical purposes, the load-carrying capacity varies as the loaded area; that is to say, as the diameter is increased the deflexion remains constant if the unit pressure is kept constant, and the effect of perimeter shear becomes relatively insignificant. For this reason also a 30-inch-diameter plate is normally adopted for subgrade bearing capacity tests.

Figs 4



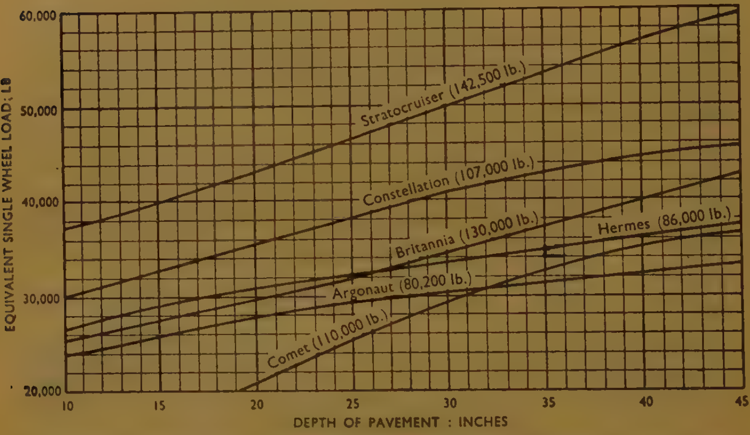
LOAD CARRIED ON A 30-INCH-DIAMETER PLATE AT VARIOUS DEPTHS OF SUBGRADE



Flexible Pavements: Variation of Equivalent Single Wheel Load with Depth of Subgrade

In *Figs 4* are shown, for a typical twin-wheel undercarriage, the loadings on a 30-inch-diameter plate in different positions vertically below a line joining the centres of the contact areas, for various depths of subgrade. The value at each point is computed from formula (1) modified for a concentration factor of 2, using a graphical method and influence diagrams similar to those developed by Newmark[7] for the Boussinesq formula. The modified Newmark diagrams give the actual intensity of vertical stress at various points on the subgrade, and the total load on the plate is obtained by summation. The maximum load on a 30-inch-diameter plate is readily obtained from the curve; and the single wheel load with the same contact pressure as the twin-wheel assembly applied on the surface, which gives the same total load on a 30-inch-diameter plate with its centre on the vertical axis of the loaded area, is the equivalent single wheel load for the assembly. The values for the equivalent single wheel load are indicated for each depth of subgrade in *Figs 4*.

Fig. 5



Flexible Pavements: Variation of Equivalent Single Wheel Load with Pavement Depth. Figures in Brackets are the assumed All-up Weights of the Aircraft

The same method is used for the assessment of the equivalent single wheel load for twin-tandem-wheel undercarriages. The process in this case is complicated by the introduction of a second horizontal dimension, and the location of the search plate for maximum superimposed load may not be on a line joining any two wheels of the assembly. Accordingly, it is necessary to draw curves of superimposed load on the search plate for several parallel lines from the line joining the centres of the outer pair of wheels to the centre-line of the assembly.

The equivalent single wheel load has been assessed by this method for several typical multiple-wheel undercarriages for various all-up weights and corresponding tire pressures. Curves showing the variation of the value with the thickness of flexible pavement are given in *Fig. 5* for several typical aircraft.

RIGID PAVEMENTS

A rigid pavement is said to fail when cracking of the material—normally concrete—of which it is constructed occurs, and cracking is nearly always caused by excessive flexural stress in the plane of the crack. The flexural stress is, therefore, the critical effect, and the equivalent single wheel load is that load which produces the same maximum flexural stress as that produced by the multiple-wheel undercarriage being considered on any particular pavement, the single wheel having the same contact pressure as the maximum contact pressure of the multiple-wheel assembly.

In Great Britain, the majority of rigid airfield pavements consist of unreinforced-concrete slabs, without dowels or other load-transfer devices. The slab corners are, accordingly, the weakest parts of the structure and failure normally commences with corner cracking. In the design of such pavements, the corner case is taken for the calculation of the thickness and flexural strength required. Logically, therefore, the corner case should be used as the basis for assessment of the equivalent single wheel load. Assuming that the contact area of the load is circular, Westergaard[1] gives the following formula for the tensile stress due to the load at the corner :

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)^{0 \cdot 6}\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where σ_c denotes tensile stress (lb. per square inch),
P ,, total load (lb.),
h ,, depth of slab (inches),
a ,, radius of contact area (inches),
and l ,, radius of relative stiffness of slab (inches)

$$= \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

where E denotes the modulus of elasticity of the concrete (lb. per square inch),
μ ,, ,, Poisson's Ratio for the concrete,
and k ,, ,, the modulus of subgrade reaction of the slab foundation (lb. per square inch per inch).

In the case of a point load when a is zero, formula (2) becomes :

$$\sigma_c = \frac{3P}{h^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

This formula is the same as that for the corner of a slab receiving no support from the subgrade, which is a condition which could only arise under extreme upward temperature curling or subgrade deterioration. Such extremes are unlikely, and formula (2) usually gives excessive values for the stress. It is, however, apparent that a breakdown of surface sealing, which would permit water to reach the subgrade, would result in seasonal, and perhaps daily, variation of the k-value and, consequently, variations of the flexural stress under the same loading conditions.

In addition, it will be noted that complications are introduced when a is greater than $\frac{l}{\sqrt{2}}$ and $\left(\frac{a\sqrt{2}}{l}\right)^{0\cdot6}$ exceeds unity. With large multiple-wheel undercarriages the contact area of the equivalent single wheel may be considerable and the a-value may, accordingly, approach the limiting value of $\frac{l}{\sqrt{2}}$.

These considerations indicated that the corner case might not prove to be satisfactory as a basis for investigation of the equivalent single wheel load relationship. Subgrade conditions also affect consideration of the edge case in the same manner, although to a less extent.

In the centres of slabs, variations of moisture content and subgrade reaction arising from faulty joint sealing are at the minimum, and variation of the equivalent single wheel load values is also at the minimum for the same reason. Accordingly, the centre-case stressing conditions have been adopted for the method of calculating equivalent single wheel loads for rigid pavements put forward in this Paper.

For the undercarriage, the variables involved are the spacing of the wheels (" undercarriage geometry "), the load on the wheels, and the tire pressure. For the pavement, the variables are the depth of the concrete slab, the modulus of elasticity and Poisson's Ratio for the concrete, and the modulus of subgrade reaction. Formula (3) combines the pavement variables in a single expression for l and, after experiments with several methods of presenting curves for computing the equivalent single wheel load, it was found easiest to adopt the two factors of $\frac{A}{l^2}$ and $\frac{S}{l}$ to give a third value which represents the ratio of the total load on the undercarriage to the equivalent single wheel load. In these factors A denotes the total contact area of the wheels of the undercarriage, and S the centre-to-centre spacing of the wheels. For twin-tandem wheels it is necessary to consider two $\frac{S}{l}$ factors, which will be discussed later.

To facilitate the derivation of the value of l for the particular pavement being considered, a Table of l-values for various combinations of E, h, and k is given in Table 1.

TABLE 1.—RIGID PAVEMENT : TABLE OF l-VALUES

Thickness of pavement h in inches	$k = 50$	$k = 100$	$k = 150$	$k = 200$	$k = 250$	$k = 300$	$k = 350$	$k = 400$	$k = 500$	$k = 1{,}000$
6	36·8	31·0	28·0	26·0	24·6	23·5	22·6	21·9	20·7	17·4
6½	39·1	32·9	29·7	27·7	26·2	25·0	24·0	23·3	22·0	18·5
7	41·3	34·8	31·4	29·3	27·6	26·4	25·4	24·6	23·3	19·6
7½	43·5	36·6	33·1	30·8	29·1	27·8	26·8	25·9	24·5	20·6
8	45·7	38·4	34·7	32·3	30·6	29·2	28·1	27·2	25·7	21·6
8½	47·8	40·2	36·3	33·8	32·0	30·6	29·4	28·5	26·9	22·6
9	49·9	42·0	37·9	35·3	33·4	31·9	30·7	29·7	28·1	23·6
9½	52·0	43·7	39·5	36·8	34·8	33·2	32·0	30·9	29·2	24·6
10	54·0	45·4	41·1	38·2	36·1	34·5	33·3	32·1	30·4	25·6
10½	56·0	47·1	42·6	39·6	37·5	35·8	34·5	33·3	31·5	26·5
11	58·0	48·8	44·1	41·0	38·8	37·1	35·7	34·5	32·6	27·5
11½	60·0	50·5	45·6	42·4	40·1	38·3	36·9	35·7	33·7	28·4
12	62·0	52·1	47·1	43·8	41·4	39·6	38·1	36·8	34·8	29·3
12½	63·9	53·7	48·5	45·2	42·7	40·8	39·3	38·0	35·9	30·2
13	65·8	55·3	50·0	46·6	44·0	42·0	40·5	39·1	37·0	31·1
13½	67·7	56·9	51·4	47·9	45·3	43·2	41·6	40·2	38·1	32·0
14	69·6	58·5	52·8	49·2	46·5	44·4	42·8	41·3	39·1	32·9
14½	71·4	60·1	54·2	50·5	47·7	45·6	43·9	42·4	40·2	33·8
15	73·2	61·6	55·6	51·8	49·0	46·8	45·0	43·5	41·2	34·7
15½	75·0	63·1	57·0	53·1	50·2	48·0	46·1	44·6	42·2	35·5
16	76·8	64·6	58·4	54·4	51·4	49·1	47·2	45·7	43·2	36·3

The values of l given in the Table must be adjusted for different values of E, as follows :—

E	Multiply l by
4,000,000	0·947
3,000,000	0·881
2,000,000	0·796

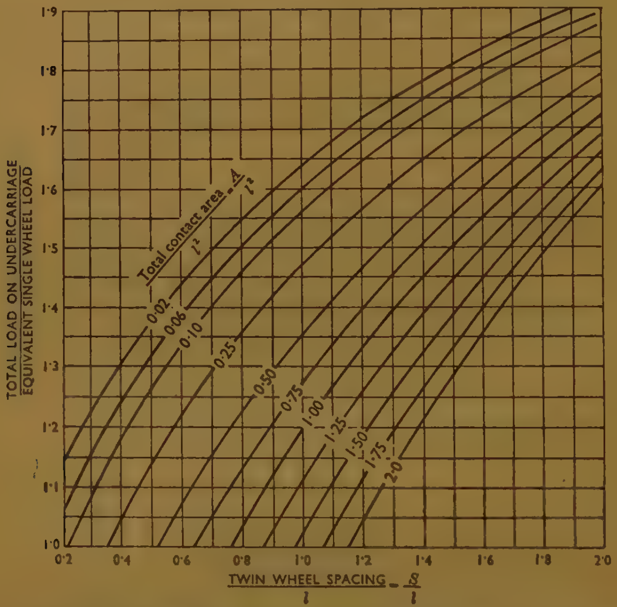
Radius of relative stiffness, $l = \sqrt{\dfrac{Eh^3}{12(1 - \mu^2)k}}$

where $\mu = 0{\cdot}15$,
$E = 5{,}000{,}000$ lb. per square inch,
k denotes modulus of subgrade reaction.

The Twin-Wheel Case

Fig. 6 shows a set of curves for the derivation of equivalent single wheel load factors for twin-wheel undercarriages. As an example, consider the case of an undercarriage carrying a total load of 49,000 lb. on wheels with a tire pressure of 92 lb. per square inch, the centre-to-centre spacing of the wheels being 30 inches. The equivalent single wheel load is required on a concrete pavement 10 inches thick, the modulus of subgrade reaction being 250 lb. per square inch per inch and the modulus of elasticity and Poisson's Ratio for the concrete 4,000,000 lb. per square inch and 0·15 respectively.

Fig. 6



RIGID PAVEMENTS: EQUIVALENT SINGLE WHEEL LOAD FOR TWIN WHEEL UNDERCARRIAGE

From Table 1:

$$l = 34 \text{ inches}, \qquad l^2 = 1{,}156 \text{ square inches}$$

$$A = \frac{49{,}000}{92} = 534 \text{ square inches}$$

$$\frac{A}{l^2} = \frac{534}{1{,}156} = 0{\cdot}46$$

$$\frac{S}{l} = \frac{30}{34} = 0{\cdot}88.$$

The ratio of total load to equivalent single wheel load is read at 1·31, and :

$$\text{E.S.W.L.} = \frac{49{,}000}{1{\cdot}31} = 37{,}400 \text{ lb.}$$

For the construction of the curves, use was made of Westergaard's [1] centre-case formula, as re-expressed by Bradbury [8] for Poisson's Ratio for concrete of 0·15 :

$$\sigma_i = 0{\cdot}31625 \frac{P}{h^2}\left[4 \log_{10}\left(\frac{l}{b}\right) + 1{\cdot}0693\right] \quad . \quad . \quad . \quad . \quad (5)$$

where P, h, and l have the same meanings as before and σ_i denotes the maximum tensile stress on the bottom of the slab on the axis of the load P.

$$b = \sqrt{1{\cdot}6a^2 + h^2} - 0{\cdot}675h \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where a, the radius of the loaded area, is less than $1{\cdot}724h$, and

$$b = a \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where a is greater than $1{\cdot}724h$

Formula (5) can be reduced to :

$$\sigma_i = \frac{CP}{h^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

where C is a stress coefficient determined by the ratio $\frac{l}{b}$. To simplify the calculation it has been assumed that $b = a$ throughout. The errors introduced by this assumption are small and are largely eliminated in the calculations. The coefficient C depends, therefore, on the ratio $\frac{l}{a}$.

Formula (8) may be re-expressed as

$$C = \frac{\sigma_i h^2}{P} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

which becomes

$$C = \frac{Mh^2}{P\frac{h^2}{6}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

since $\sigma_i = \frac{M}{\left(\frac{h^2}{6}\right)}$, where M denotes the bending moment and $\frac{h^2}{6}$ the section modulus per unit length of the slab. Accordingly

$$\frac{M}{P} = \frac{C}{6} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

A Table of values for C for various values of $\frac{l}{b}$ has been prepared by E. F. Kelley.[9]

The data presented for the estimation of an equivalent single wheel load for a particular twin-wheel undercarriage on a particular pavement are the total load P_1, the tire pressure p, the centre-to-centre spacing of the wheels S, and the pavement characteristics, summarized in the expression l. If P_2 represents the equivalent single wheel load, also at a tire pressure p, the ratio $\frac{P_1}{P_2}$ is required for any combination of the three variables p, S, and l. From formula (5) it is apparent that the stress varies as the applied load if the area of application of the load is kept constant, and it is therefore convenient to adopt the total contact area of the twin-wheel undercarriage A_1 as a variable instead of the tire pressure, and $A_1 = \frac{P_1}{p}$. A practical set of curves can be constructed by expressing the dimensions A_1 and S in terms of l, thus combining the three variables to give two factors $\frac{A_1}{l^2}$ and $\frac{S}{l}$ from which the values of $\frac{P_1}{P_{\text{[illegible]}}}$ can be obtained.

Values of $\frac{A_1}{l^2}$ in the range 0·02 to 2·00, and for $\frac{S}{l}$ in the range 0·25 to 1·75, were selected. These ranges cover all practical variations of the three dimensions involved.

Commencing with the values of $\frac{A_1}{l^2}$, then, since $A_1 = 2\pi a_1{}^2$ where a_1 represents the radius of the contact area of each wheel of the undercarriage, corresponding values of $\frac{l}{a_1}$ were obtained. The appropriate values for C were then taken from Kelley's Tables and, from formula (11), the corresponding $\frac{M_1}{0{\cdot}5P_1}$-values were derived. M_1 represents the bending moment directly under the first wheel being considered, and the load value is the load on one wheel, that is to say, $0{\cdot}5P_1$.

Westergaard has given curves[1] for the estimation of $\frac{M}{P}$-values due to a point load at any distance from the point of application of the load, expressed in terms of l. The values given are the tangential and radial values, the former being the higher in every case. The $\frac{M_1}{0{\cdot}5P_1}$-values already obtained under the first wheel are radial and equal in all directions, therefore the tangential values for the second wheel $\frac{M_2}{0{\cdot}5P_1}$ were added to give the total $\frac{M}{0{\cdot}5P_1}$-values under the first wheel, for the various selected values of $\frac{S}{l}$. At this stage values of $\frac{M}{0{\cdot}5P_1}$ were available for a range of values of $\frac{A_1}{l^2}$ and $\frac{S}{l}$.

An arbitrary value was selected for the tire pressure p. This was assumed at 100, and multiplying the $\frac{A_1}{l^2}$-values by this figure gave the corresponding $\frac{P_1}{l^2}$-values, that is to say, values for the total load on the undercarriage in terms of l^2. Multiplying the $\frac{M}{0{\cdot}5P_1}$-values by the appropriate $\frac{0{\cdot}5P_1}{l^2}$-values gave the corresponding $\frac{M}{l^2}$-values, or values for the total bending moment in terms of l^2.

It then remained to determine the equivalent single wheel load P_2 which, at the same arbitrary tire pressure of 100, gave the same M-values, in terms of l^2. This was done by reversing the calculations, again making use of Kelley's Table and formula (5). Values for the required ratio of $\frac{P_1}{P_2}$ for each value of $\frac{A_1}{l^2}$ and $\frac{S}{l}$ were then obtained by dividing the $\frac{P_1}{l^2}$ values by the $\frac{P_2}{l^2}$ values, the l^2 factor, since it is constant, being eliminated in each case.

It will be noted that any figure may be selected for the tire pressure; its introduction may in fact be omitted from the calculations, but it assists in the explanation of the method.

Examples of the calculation are given in Table 2.

The Twin-Tandem Case

Fig. 7 shows a set of curves for the derivation of equivalent single wheel load factors for twin-tandem undercarriages. As an example consider the case of a four-wheel undercarriage carrying a total load of 102,000 lb. with a tire pressure on all four wheels of 70 lb. per square inch, the wheel base of the bogie being 60 inches and the track 31 inches. The equivalent single wheel load is required on a concrete pavement 12 inches thick, the modulus of subgrade reaction being 150 lb. per square inch per inch, and the modulus of elasticity and Poisson's Ratio for the concrete 4×10^6 lb. per square inch and 0·15 respectively.

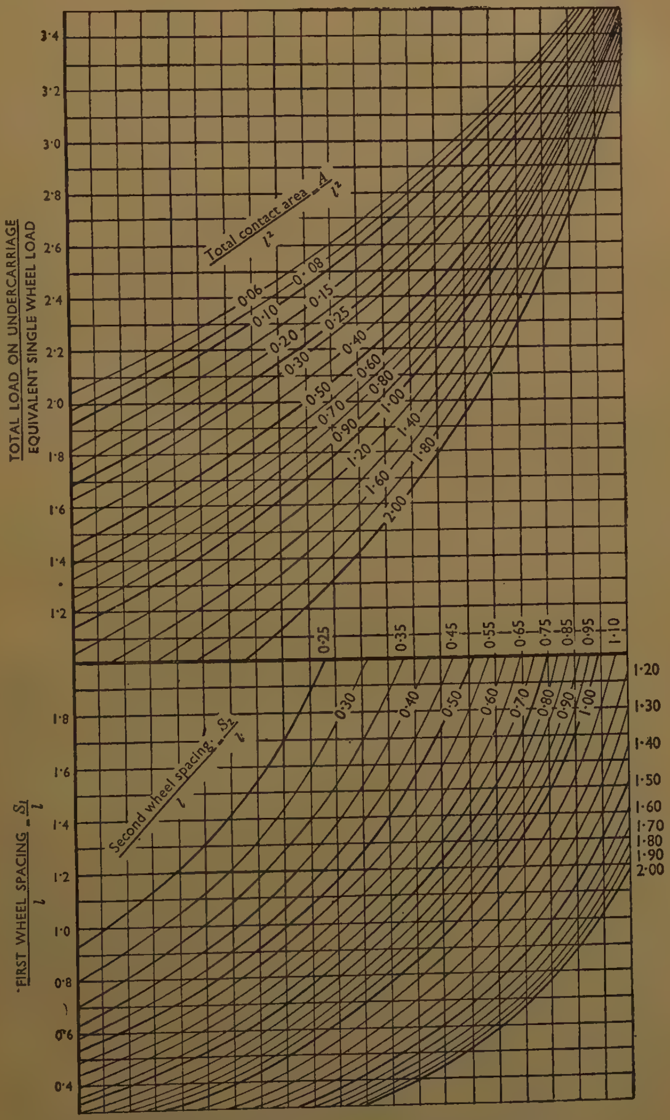
From Table 1: $l = 44{\cdot}5$ inches; $l^2 = 1{,}980$ square inches

$$A = \frac{102{,}000}{70} = 1{,}450 \text{ square inches}$$

TABLE 2.—TYPICAL CALCULATIONS FOR PREPARATION OF EQUIVALENT SINGLE WHEEL LOAD CURVES FOR RIGID PAVEMENTS: TWIN-WHEEL UNDERCARRIAGES

Line	Formula	Interpretation	How obtained	Examples	
1	$\frac{A_1}{l^2}$	Total contact area of twin-wheel undercarriage ÷ l^2	Data	0·400	1·00
2	$\frac{l}{a_1}$	l ÷ radius of contact area of one wheel	$\frac{1}{\sqrt{\frac{(\text{line 1})}{2\pi}}}$	3·95	2·51
3	C	—	Kelley's Table	1·093	0·845
4	$\frac{M_1}{0{\cdot}5P_1}$	Bending moment under first wheel due to load on first wheel ÷ load on first wheel	$\frac{(\text{line 3})}{6}$	0·182	0·141
5	$\frac{P_1}{l^2}$	Total load on twin-wheel under-carriage ÷ l^2	100 × (line 1)	40·0	100·0
6	$\frac{S}{l}$	Centre-to-centre spacing of wheels ÷ l	Data	0·75	1·25
7	$\frac{M_2}{0{\cdot}5P_1}$	Bending moment under first wheel due to load on second wheel ÷ load on second wheel	Westergaard's Chart	0·075	0·038
8	$\frac{M}{0{\cdot}5P_1}$	Total bending moment under one wheel due to loads on both wheels ÷ load on one wheel	(line 4) + (line 7)	0·257	0·179
9	$\frac{M}{l^2}$	Total bending moment due to both wheels ÷ l^2	0·5 × (line 5) × (line 8)	5·14	8·95
10	$\frac{P_2}{l^2}$	Equivalent single wheel load ÷ l^2	Reverse calculation	31·8	72·5
11	$\frac{P_1}{P_2}$	Total load on twin-wheel undercarriage ÷ equivalent single wheel load	(line 5) ÷ (line 10)	1·26	1·38

Fig. 7



RIGID PAVEMENTS: EQUIVALENT SINGLE WHEEL LOAD FOR TWIN-TANDEM UNDERCARRIAGES

$$\frac{A}{l^2} = \frac{1,450}{1,980} = 0{\cdot}73$$

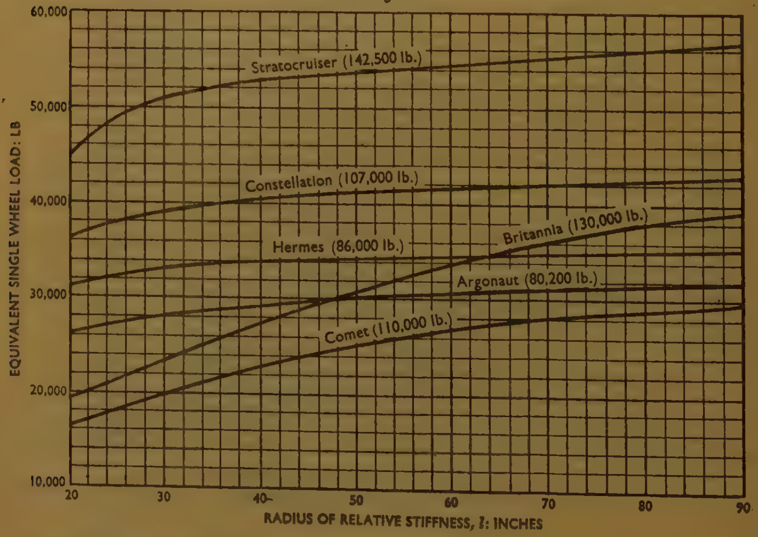
$$\frac{S_1}{l} = \frac{60}{44{\cdot}5} = 1{\cdot}35$$

$$\frac{S_2}{l} = \frac{31}{44{\cdot}5} = 0{\cdot}70$$

where S_1 and S_2, represent the two main undercarriage dimensions. The $\frac{S_1}{l}$ and $\frac{S_2}{l}$ factors are combined on the set of curves at the foot of the figure, and the point determined is carried vertically upwards to meet the appropriate $\frac{A}{l^2}$ value in the upper set of curves, where the reduction factor of 2·37 is obtained. It will be noted that the first and second wheel spacing factors $\frac{S_1}{l}$ and $\frac{S_2}{l}$ may be interchanged to give the same result. The equivalent single wheel load is then $\frac{102,000}{2{\cdot}37} = 43,000$ lb.

The construction of the curves is carried out in the same way as the construction of the curves for the twin-wheel assembly. The total $\frac{M}{P}$-value

Fig. 8



Rigid Pavements: Variation of Equivalent Single Wheel Load with Pavement-stiffness "l." Figures in Brackets are the Assumed All-up Weights of the Aircraft under one wheel of the assembly is obtained by adding the effect of the other three wheels to that of the wheel being considered, and, in this instance, the principal stress has to be investigated in detail for each case in view of the obliquity of the wheel diagonally opposite that being considered.

A set of curves for the derivation of the equivalent single wheel load for concrete pavements with different l-values, for several typical multiple-wheel-undercarriage aircraft, is given in *Fig. 8.*

OTHER VARIABLES—UNDERCARRIAGES

The methods which have been described above for the evaluation of equivalent single wheel loads cover dual and twin-tandem undercarriages, and the following assumptions have been made :

(*a*) that the tire pressures of all the wheels of the assembly are identical, and
(*b*) that the lay-out of the four wheels is symmetrical in the case of twin-tandem undercarriages.

Where condition (*a*) is not met, it has been found that a reasonable degree of accuracy is obtained by taking the tire pressure as the highest of the group of wheels, and, when condition (*b*) is not met, by taking the minimum value for the track dimension. If there is any doubt concerning the accuracy of results obtained in this manner, the equivalent single wheel load for the pavement under consideration must be calculated from first principles by the method outlined for the compilation of the curves.

Another type of undercarriage, which is sometimes used on large aircraft, or where it is necessary to reduce the effect of the wheel loading to a considerable extent to permit operation off grass surfaces, employs two wheels side by side to replace each single wheel of a normal dual or twin tandem undercarriage. In such cases it is reasonable to assume that the contact area is the circle of minimum radius which will include the contact area of the pair of wheels, and assess the tire pressure as the load on the pair of wheels divided by the area of the circle. The equivalent single wheel load assessment is then made as for a normal dual or twin-tandem undercarriage, the centres of the contact areas being regarded as being on the axis of each pair of wheels, and the load on that contact area as the total load on the pair ; that is to say, one-half or one-quarter of the total load on the assembly in the case of dual or twin-tandem undercarriages respectively. As in cases (*a*) and (*b*) above, the equivalent single wheel load must be calculated from first principles if there is any doubt regarding the accuracy of the result given after making the assumption regarding contact area.

Other Variables—Pavements

It has been remarked on pp. 423 and 429 that subgrade failure is taken as the factor to be considered for the assessment of equivalent single wheel loads on flexible pavements, and cracking of the concrete for the assessment of E.S.W.L. on rigid pavements. Rigid pavements, in view of their homogeneous nature throughout their depth, are unlikely to fail in any other manner but, in flexible pavements, it may happen that failure can occur in an intermediate layer of the base course. This is unlikely in a well-designed pavement where the base courses from the subgrade upwards have a reasonable reserve of strength, but it is necessary to investigate the possibility of such failure in detail if the strength of any intermediate layer in a deep flexible pavement is suspect.

In a natural unsurfaced grass airfield, the bearing value of the soil normally increases with depth instead of diminishing as in a flexible pavement. Since the total vertical pressures caused by the wheels of a multiple-wheel assembly at any depth must always be less than the pressure under one wheel at the surface, if the wheels are reasonably well spaced, it follows that, on a natural unpaved airfield, the equivalent single wheel load for the assembly is the load on one individual wheel of the assembly. If the total load is unequally distributed over the wheels, the equivalent single wheel load is that of the wheel carrying the most adverse combination of loading and tire pressure.

Temporary airfields surfaced with metal tracking, such as "Pierced Steel Plank," present a particularly difficult problem. As yet, no altogether satisfactory method of design has been devised for such pavements. If a single wheel is replaced by a dual undercarriage and the contact areas of the two wheels are wholly contained on one sheet of the surfacing, the effect of the loading is very little less than that of the single wheel at the same tire pressure which, in any case, would probably have a contact area sufficiently great to overlap on to the adjacent sheet. A further factor is that metal tracking is normally found on natural surfaces of doubtful behaviour. On airfields of this type consideration of the effect from first principles is unavoidable.

Design and Evaluation

It will be apparent from the above that determination of the equivalent single wheel load is the first requirement both for design and evaluation. Trial and error methods are required unless design or evaluation curves are available for the particular undercarriage being considered.

In the design of flexible pavements, a construction depth is assumed for the design subgrade bearing value and, with experience, the first assessment usually proves to be accurate. For this depth the equivalent single wheel load is assessed and the thickness is checked to ensure that it is sufficient for the load. In the detail design, consideration has to be given to the effect of intermediate base course layers as just described.

In the C.B.R. method of flexible pavement design,[2] design curves have been prepared by the Office of the Chief of Engineers, U.S. Army, for particular undercarriage arrangements.

For the design of rigid pavements, trial-and-error methods are adopted for the corner case in undowelled slabs. The Office of the Chief of Engineers,[10] U.S. Army, has developed rigid pavement design curves for dowelled slabs for particular undercarriage arrangements, and a method has also been developed by the Portland Cement Association,[11] using Professor Pickett's [12] influence charts.

A design method for multiple wheel undercarriages on a " sandwich " pavement, consisting of a depth of flexible construction on a concrete base-slab, has been developed by De Kruyf, Poel & Timman.[13]

For evaluation purposes the majority of airfields in the United Kingdom have been assigned a Load Classification Number. On any individual airfield the equivalent single wheel load for a specific multiple-wheel undercarriage can be determined, and from this load and its associated tire pressure the Load Classification Number [3] of the aircraft can be determined. It will readily be appreciated that any multiple-wheel undercarriage aircraft has a different Load Classification Number for each airfield it uses, depending upon the equivalent single wheel load. When it is necessary to determine whether or not a particular multiple-wheel undercarriage aircraft may operate from an airfield, the equivalent single wheel load must first be assessed with respect to the airfield pavement; the Load Classification Number obtained from this load value is then compared directly with that of the airfield.

Other Methods

Much work on the multiple-wheel undercarriage problem has been done by the United States Corps of Engineers. Full-scale studies were carried out by this authority on the effect of the dual-wheel undercarriage of the B-29 aircraft, now used by the Royal Air Force and known as the Washington bomber. This aircraft was, in 1943, the heaviest aircraft known and the first of considerable size to employ such an undercarriage. This study provided much information of interest and details have been published.[14]

The study on the B-29 aircraft led to the publication by C. R. Foster [15] of a method for assessing equivalent single wheel loads for flexible pavements. The method assumes that inter-action commences at a depth below the surface of the pavement equal to one-half the clear distance between the contact areas of the most closely-spaced wheels, and is complete at a depth equal to twice the distance between the axes of the most widely-spaced pair of wheels. Through this range the equivalent single wheel load varies between that of an individual wheel and the total load on the assembly; it remains constant at the load on an individual wheel and the total load on the assembly beyond these limits. Between the limits it is assumed that the equivalent single wheel load varies in a logical manner, and, to obtain the value at any depth, a straight line is drawn between the limiting values on a log-log plot of depth against equivalent single wheel load.

Values given by the Corps of Engineers method for flexible pavements are slightly higher in the shallower pavements than those given by the method described in this Paper, and lower in the deeper pavements.

The Corps of Engineers method referred to takes only limited account of tire pressure, but further work has been done by C. R. Foster at the Waterways Experimental Station, Vicksburg, Mississippi, which allows for this factor. This is not published as yet.

The Corps of Engineers has also developed a method for evaluation of equivalent single wheel loads on rigid pavements. In this work, which was carried out by E. P. Bone of the Rigid Pavement Laboratories, Cincinatti, Ohio, and is not yet published, the edge case is considered and it is assumed that load transfer devices result in a 25 per cent decrease of bending moment in the edge of the concrete slab. The problem is approached in the same manner as that described in this Paper, and the equivalent single wheel load assessment curves are presented similarly in terms of $\frac{A}{l^2}$ and $\frac{S}{l}$. However, for twin-tandem assemblies, it is assumed that the second pair of wheels can be ignored if the track dimension exceeds the value of l for the pavement under consideration.

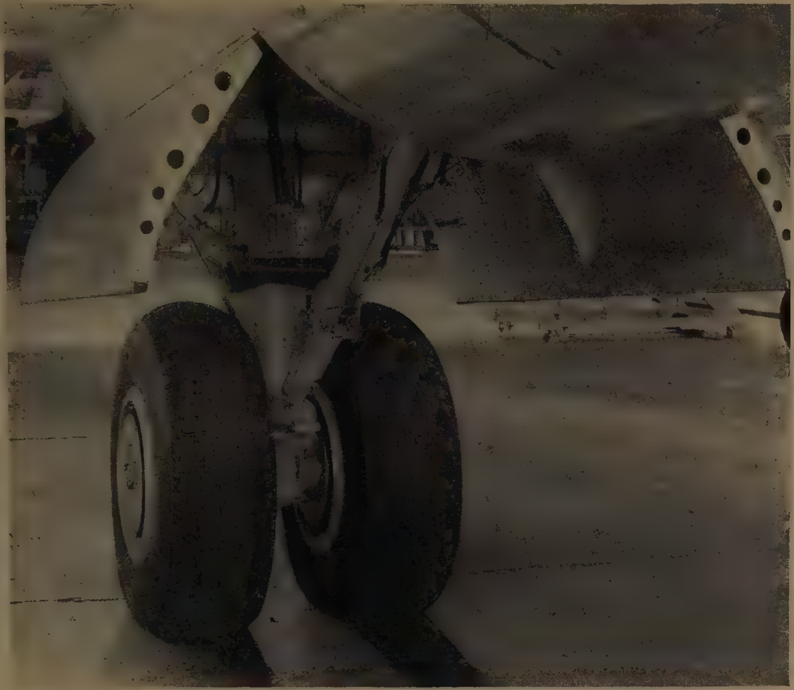
Results for rigid pavements obtained from the Corps of Engineers methods correspond closely to those given by the method described in this Paper, the range of variation being about 10 per cent.

Another method, developed by S. Rodin,[16] is used by the British Overseas Airways Corporation. This method is based on the Boussinesq theory, and is only applicable to flexible pavements. It assumes that the pavement is homogeneous throughout its depth, a theoretical condition which is unlikely to obtain in practice. In this method the deflexion on the surface due to the multiple-wheel assembly is calculated, and the equivalent single wheel load is that on a single wheel which produces the same deflexion. The method takes no account of different types and depths of pavement construction.

FURTHER INVESTIGATIONAL WORK

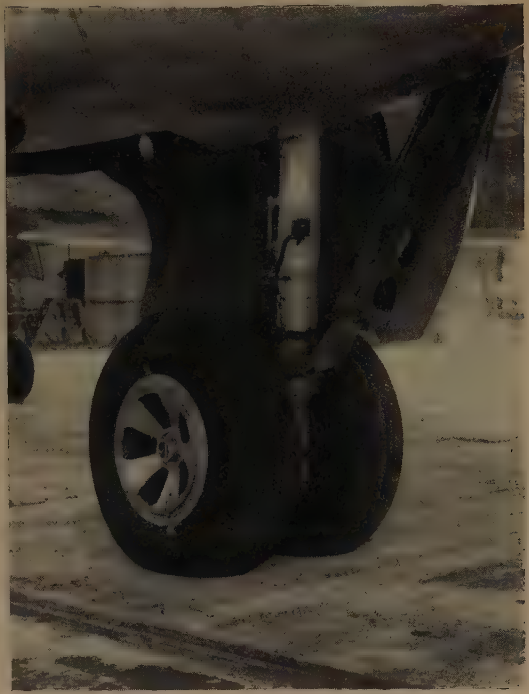
A considerable amount of experimental testing has been carried out by the Air Ministry, particularly on rigid pavements, using single and multiple bearing-plates in a 100-ton testing bridge. The examination and analysis of the data obtained from these tests has called for extensive

Fig. 9



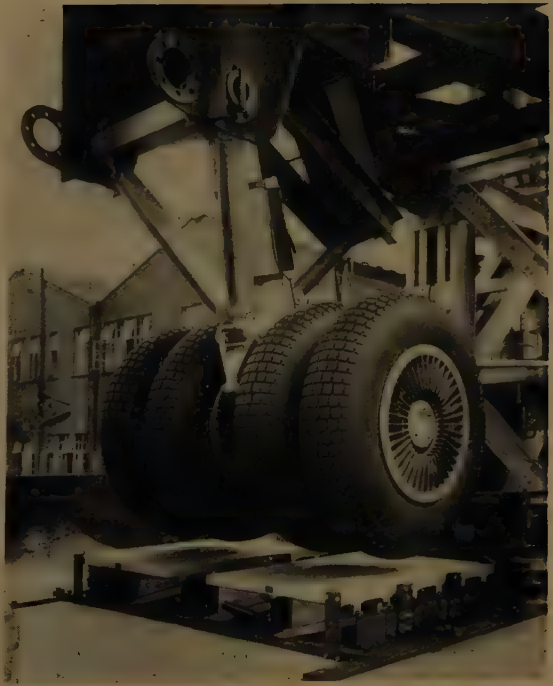
Handley Page Hermes Mk 4 Undercarriage

Fig. 10



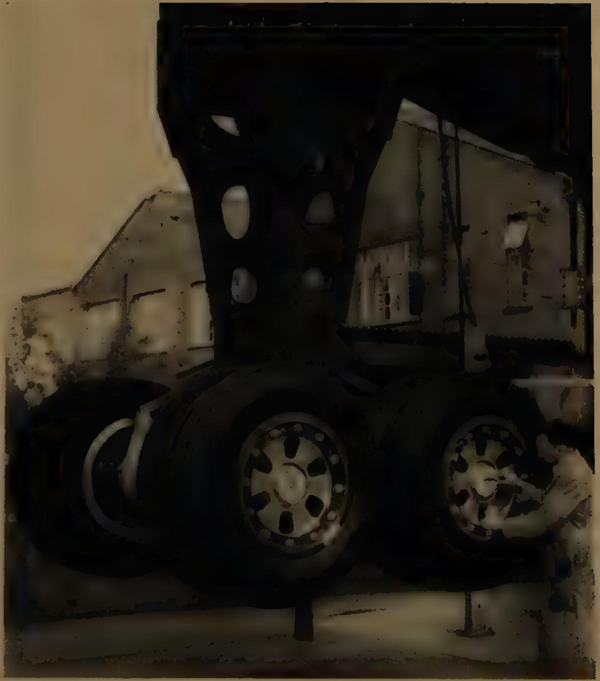
Vickers Viscount Undercarriage

Fig. 11



Bristol Brabazon Mk. I Undercarriage

Fig. 12



Bristol Brabazon Mk 2 Undercarriage

study, and the conclusion has been reached that full-scale tests on existing pavements are subject to so many variable external influences as to be of little value. For example, the load sufficient to fracture the corner of a concrete slab under four bearing-plates, representing a twin-tandem undercarriage, might be found, but it can never be absolutely certain that subgrade conditions, etc., are identical under the corner to which a single plate is applied to determine the equivalent single wheel load. In addition, the single- and multiple-plate tests should be carried out simultaneously to ensure that temperature and atmospherical moisture conditions, which can have a marked effect on the bearing value of a concrete slab, are identical.

These factors have led to the conclusion that large-scale model-tests, under controlled laboratory conditions, are required to produce satisfactory results and, accordingly, arrangements have been made for such tests to be carried out on both flexible and rigid pavements at two universities in Great Britain. These investigations will necessarily take a considerable time to complete and results cannot be expected for several years.

TYPICAL UNDERCARRIAGES

Figs 9 to *15* are photographs of some typical British multiple-wheeled undercarriages. Brief descriptions of these follow, with special reference to their effects on airfield pavements.

Fig. 9 shows the undercarriage mounted on the Handley Page Hermes aircraft. This undercarriage is a straightforward dual assembly which presents no particular difficulties. *Fig. 10* shows the same type of undercarriage mounted on the Vickers Viscount.

Fig. 11 shows the undercarriage mounted on the Bristol Brabazon Mark I. This undercarriage does not fall into either of the normal categories, and each pair of wheels on either side of the undercarriage leg has to be compounded to an equivalent contact circle by the method described on pp. 440–441, before the equivalent single wheel load assessment is made as for a standard dual-wheel undercarriage. *Fig. 12* shows the undercarriage proposed for the Brabazon Mark II, where the same process is required to reduce the data to that of a normal twin-tandem undercarriage.

Fig. 13 shows the twin-tandem undercarriage mounted on the General Aircraft Universal Freighter, an aircraft specially designed with a very low tire pressure to operate off pavements of low bearing value. This is a straightforward undercarriage for which the equivalent single wheel load for any particular pavement can be obtained direct from the curves.

Fig. 14 shows the twin-tandem undercarriage of the De Havilland Comet. This undercarriage has differing spacings for the front and rear wheels, and the lesser dimension is taken for the assessment of the equivalent single wheel load.

Fig. 15 shows the twin-wheel undercarriage as used on the Vickers

Valiant jet-engined bomber. This undercarriage differs from the normal dual-wheel undercarriage as the two wheels are arranged in tandem instead of side by side. The effect does not differ from that of the normal arrangement, the critical conditions as calculated not being altered by the orientation of the wheels. The lay-out assists the aircraft designer by reducing the thickness of the assembly and facilitating housing in thin wing sections, and the equivalent single wheel load is relatively low in view of the greater distance between the axes of the wheels. A point to be borne in mind with this type of undercarriage, and also with twin-tandem undercarriages which are fixed longitudinally, is the side force exerted in turning. The stresses set up considerably affect the design of the tire and the undercarriage supporting structure, and the airfield pavement designer must take into account the resistance of the runway surface to horizontal shear. No difficulty arises in unsurfaced concrete pavements, but asphalt surfaces require a high degree of stability.

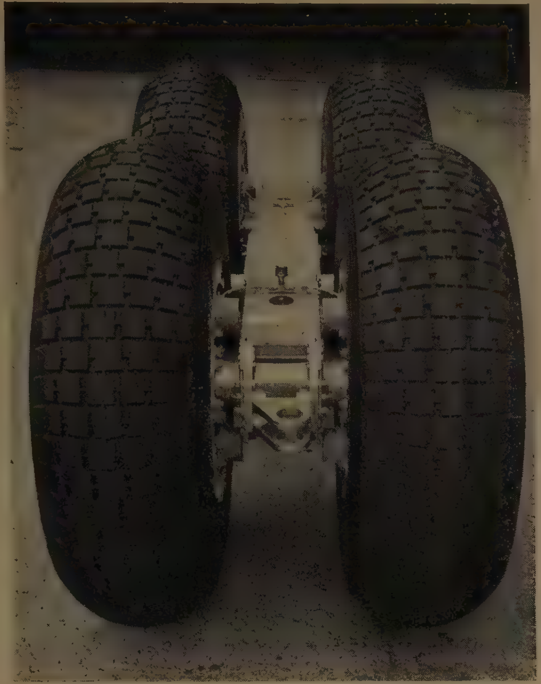
APPLICATION AND LIMITATIONS

The multiple-wheel undercarriage does not present an insoluble problem to the airfield pavement designer. If his design criteria include a list of all aircraft likely to use the pavements to be designed, he can very readily select the two or three types, single- or multiple-wheel, which are likely to give the most adverse effect. On a subgrade of high bearing value requiring a relatively thin pavement, the aircraft with the heaviest individual wheel loading is likely to provide the highest stressing; conversely, on a subgrade of poor bearing value requiring a relatively thick pavement, the aircraft with the largest total load on one undercarriage is likely to provide the basis for design. The design is then prepared for one or other of the assemblies and checked for the remaining assemblies by assessment of the equivalent single wheel loads related to the design thickness. If the tire pressures vary, a comparison of Load Classification Numbers is required.

The aircraft designer's problem, however, is only partially solved. It has been the practice to specify the Load Classification Number for a projected aircraft to be manufactured, and this presents no difficulty in the case of single-wheeled aircraft. Up to a certain maximum all-up weight the aircraft designer is able to comply with the Load Classification Number requirement by adjusting the tire pressure, but above this maximum weight the tire required would be too large either for manufacture or for housing in the aircraft when retracted. Above this loading a multiple-wheeled undercarriage must be used.

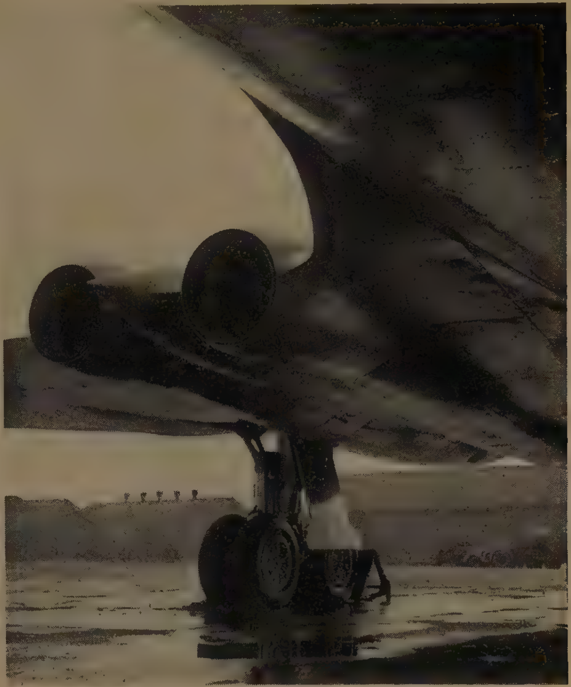
If it has been decided that a multiple wheel undercarriage is required, the aircraft designer cannot completely meet the Load Classification Number requirement unless he knows the characteristics of the pavements likely to be used. The authority ordering the aircraft may not always be

Fig. 13



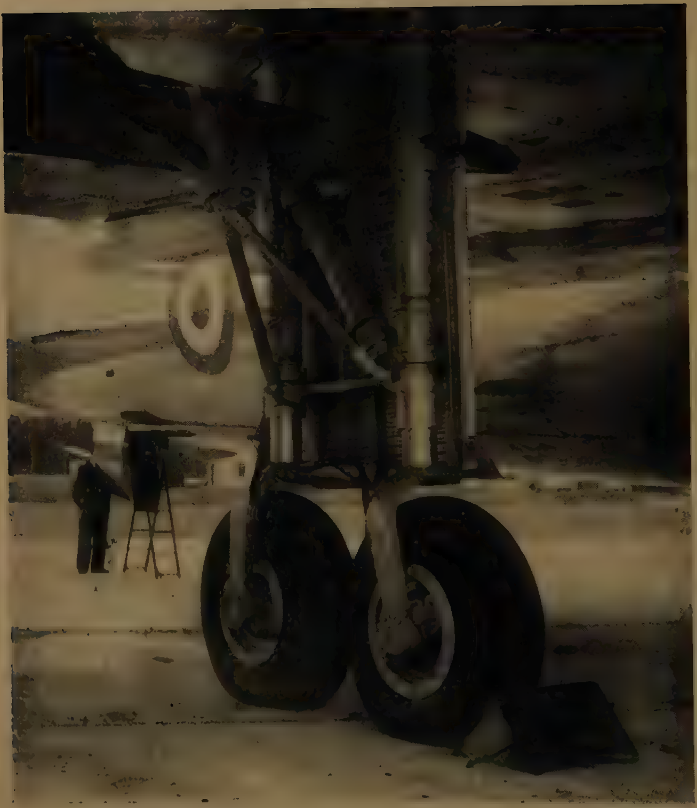
Blackburn and General Aircraft
Universal Freighter Undercarriage

Fig. 14



De Havilland Comet Undercarriage

Fig. 15



Vickers Valiant Undercarriage

able to supply this data, except perhaps in civil aviation where the route and airfields to be used are known. The data could not be supplied for military aircraft excepting sometimes in the case of very large bombers intended to operate from specified existing airfields. However, it has been shown that the equivalent single wheel load, and consequently the Load Classification Number, for a particular multiple-wheel undercarriage, increases as the construction depth of the pavement increases. Accordingly, it is logical to assess the Load Classification Number with regard to the thickest pavement from which it is likely to operate. For example, it might be specified that an aircraft is to have a Load Classification Number of not more than 30. The maximum thickness of rigid pavement required to give such a Load Classification Number would be found on the poorest possible subgrade, such as a heavy clay with a *k*-value of 100. The design thickness of a concrete pavement for this subgrade would be about 12 inches and the *l*-value, accordingly, would be about 52. With this knowledge the aircraft designer can proceed, making use of the method of evaluation put forward in this Paper, coupled with the Load Classification Number system. Any pavement thicker than that adopted for design purposes would have a higher Load Classification Number, and the rate of increase of Load Classification Number is more rapid than that of the equivalent single wheel load ; on a thinner pavement with the same or a higher Load Classification Number the equivalent single wheel load would be reduced. The aircraft could, therefore, operate safely from all airfields with Load Classification Numbers in excess of 30. It is just possible that the aircraft might also be able to operate safely from one or two airfields with Load Classification Numbers less than 30, where the subgrades have high bearing values and the pavements are in consequence relatively thin.

CONCLUSIONS

The Author has endeavoured to draw attention to the complexity of the problem. There is no simple solution to it and, at the best, results given may prove inaccurate in practice due to the many variables introduced by seasonal effects such as temperature and moisture. However, something is required, if only for guidance, and the methods given above are related to average conditions and are thought to give results sufficiently accurate for practical purposes. In any instance where, due to unorthodox methods of construction or local variations in pavement characteristics, the error is measurable, the factor of safety normally allowed in pavement design should be sufficient to cover it. At the present time, when it is usual to give considerable thought to the bearing capacity of airfields and to examine individual cases closely before permitting the use of an airfield by a particular aircraft, any damage to the pavement caused by overloading due to faulty assessment is likely to be so slight that the aircraft will not suffer, nor will operation be affected. The only result will be an additional maintenance problem for the airfield engineer.

ACKNOWLEDGEMENTS

The Paper is presented by kind permission of Mr G. H. Fretwell, C.B., M.I.C.E., Director General of Works, Air Ministry, and the Author desires to record his thanks to colleagues in the Air Ministry for their assistance in its preparation. He is particularly grateful to Messrs A. F. Smith, A.M.I.C.E., A.M.I.Struct.E., and B. M. Taylor, B.Sc., Stud.I.C.E., who were largely responsible for the development of the methods put forward in the Paper. He also wishes to record his thanks to Mr Gayle McFadden, Chief, Airfields Branch, Office of the Chief of Engineers, U.S. Army, for permission to refer to the work of the Corps of Engineers as performed by Mr Charles R. Foster and Mr Evan P. Bone.

Finally, the Author wishes to record his thanks to the following aircraft and undercarriage manufacturing companies :—

The De Havilland Aircraft Company, Limited,
Dowty Equipment, Limited,
Blackburn and General Aircraft, Limited,
Handley Page, Limited,
Vickers-Armstrongs, Limited,

for supplying, and giving permission to reproduce, the photographs of multiple-wheel undercarriages in *Figs 9* to *15*.

REFERENCES

1. H. M. Westergaard, " Computation of Stresses in Concrete Roads." Proc. 5th Annual Mtg Highway Res. Board, Washington, D.C., vol. 5, 1926.
2. Office of the Chief of Engineers, Corps of Engineers, U.S. Army, " Airfield Pavement Design : Flexible Pavements." Engng Manual, Pt XII, Ch. 2, July 1951.
3. J. A. Skinner, " Testing Runway Foundations and Pavements." Instn Civ. Engrs, Airport Paper No. 17, 1951.
4. J. Boussinesq, " *Application des Potentiels à l'Etude de l'Equilibre et du Mouvement des Solides Elastiques.*" (" The Application of Potentials to the Study of the Equilibrium and Movement of Elastic Solids.") Gauthier-Villars, Paris, 1885.
5. O. K. Fröhlich, " *Druckverteilung in Baugrunde.*" (" Earth Pressures in Building Foundation.") Springer, Vienna, 1934.
6. O. J. Porter, " Accelerated Traffic Tests at Stockton Airfield, California (Stockton Test No. 2)." U.S. Corps of Engrs, Sacramento District, 1948.
7. N. M. Newmark, " Estimating Earth Pressures." *Engng News Rec.*, vol. 120, p. 23 (Jan. 1938).
8. R. D. Bradbury, " Reinforced Concrete Pavements." Wire Reinforcement Inst, Washington, D.C., p. 31, 1938.
9. E. F. Kelley, " Application of the Results of Research to the Structural Design of Pavements." *Public Roads*, vol. 20, p. 91, 1939.
10. Office of the Chief of Engineers, Corps of Engineers, U.S. Army, " Airfield Pavement Design : Rigid Pavements." Engng Manual, Pt XII, Ch. 3, July 1951.
11. " Design of Concrete Airport Pavements." Portland Cement Association, Chicago, 1950.

12. G. Pickett and G. K. Ray, "Influence Charts for Concrete Pavements." Proc. Amer. Soc. Civ. Engrs, vol. 76, no. 12, 1950.
13. M. de Kruyf, C. v. d. Poel and R. Timman, "Calculation of Stresses in Road and Runway Constructions." Proc. 2nd Int. Conf. Soil Mech. & Foundn Engng, 1948, vol. IV, p. 181 (VIII c3).
14. Corps of Engineers, U.S. Army, Mississippi River Commission, "Certain Requirements for Flexible Pavement Design for B-29 Planes." U.S. Wtwys Exp. Stn, Vicksburg, Mississippi, August 1945.
15. C. R. Foster, "Certain Considerations in the Design of Flexible Pavements, Bases, and Subgrades." Corps of Engineers, U.S. Army, Bulletin No. 29, 1947.
16. L. J. Colyer, *Correspondence* on Airport Paper No. 17. Suppl. Engng Div. Pprs Instn Civ. Engrs, 1951.

The Paper is accompanied by seven photographs and eight sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

The Author, introducing the Paper, indicated a few misprints in the advance proof, which have since been corrected.

Mr S. Scott Hall observed that about 5 years ago the increasing size and cost of runways and the trends of aircraft design, which showed no hope of alleviating those problems, had caused great concern. Following a discussion with Mr Fretwell, the Director-General of Works in the Air Ministry, a Committee had been formed to embrace all the interests concerned and to bring together the aircraft designer on the one hand and the civil engineer responsible for the runway on the other. As Chairman of that Committee, which he thought had done some useful work during the past 5 years, Mr Scott Hall wished to take the opportunity of paying a tribute to the way in which the civil engineers received ideas from the aeronautical fraternity. Since the aeronautical engineers had been concerned only with very light structures, a certain degree of resentment on the part of the civil engineers might have been expected, but in fact they had taken the suggestions made to them in extremely good part, and, thanks to their co-operation, it had been possible to go quite a long way in dealing with the problem.

The Paper was an example of the high quality of effort which had been made by the civil engineers to deal with those problems, which were so vital to the aircraft. Any work similar to the Author's was essentially based on the assumptions which were made. It was surprising to find that nowhere in the Paper had there been a reference to the underlying basic assumption in all that work, namely that the aircraft static case was the critical case for a runway. It was not at all apparent when first approaching the problem that that was the case. There was good reason now to suppose that it was, but Mr Scott Hall still thought that in a Paper of that kind that assumption ought to be mentioned.

He also noticed that at the foot of p. 423 the assumption was made that for flexible pavements failure occurred in the subgrade before it occurred at any other level. One summer morning a pilot had landed a fairly heavy aircraft on a flexible runway, and had eaten his lunch while the aircraft stood at the end of that runway, which was not in other use. His landing had been perfectly successful, but at the end of his lunch he had found that the wheels had sunk into the tarmac to such an extent that he could not move the aeroplane. Mr Scott Hall had been assured that that was a true story and perhaps threw some doubt on the statement that failure always occurred at the subgrade and not at any other level.

Perhaps the most important point, from the aircraft designer's point of view, which had been brought out in the Paper was the fact—for such it appeared to be—that there was no longer a singular value of the Load Classification Number to be associated with a given runway. In other words, if the aircraft designer used a multiple-wheel system he had at once to break away from what it had been hoped was a fairly simple system of classification, which would tell him at once to what Load Classification Number he had to design. Mr Scott Hall was still not very clear on that point, because it seemed to him that, if that were the case, stiffness of the runway no longer implied the strength of that runway, which was rather baffling; would the Author enlarge on that? It seemed clear from what the Author had said, and from *Fig. 8*, that the simple classification had been left behind.

On p. 443 the Author had said that "the conclusion has been reached that full-scale tests on existing pavements are subject to so many variable external influences as to be of little value." Mr Scott Hall found that statement surprising because he thought that in fact the Air Ministry had been depending on those bearing-plate methods of test for the assessment of runway strengths. The Author had gone on to say "These factors have led to the conclusion that large-scale model-tests, under controlled laboratory conditions, are required to produce satisfactory results. . . ." What model tests, and how much work had been done which had led to that conclusion? If, in fact, it was not possible to deduce anything from full-scale tests on existing pavements, because of variable external influences, how could one deduce from any tests the strength under operational conditions which were themselves subject to those same external influences?

Mr Scott Hall had recently flown back from Australia, and he had been astonished to find himself landing and taking off in a Constellation, for which the take-off weight was 107,000 lb., on P.S.P. (pierced steel plank) on extremely soft ground, on extremely short runways at places such as Singapore. That had been going on for a long time, and that practical experience made him wonder whether design requirements were not too stringent.

It should be appreciated that hitherto it was only the extension of runways which had allowed the aircraft designer to obtain the high speeds and performance of modern aeroplanes. They were essentially dependent on the long runway. Now that the day had dawned when thrusts were, in fact, greater than the weight of the aeroplane he looked forward to the time, in spite of the fact that it would end their happy association with civil engineers, when runways would be unnecessary.

Mr K. C. Mann said that he believed that the Author's Paper was the first to throw light upon a major problem and to give a practical method of solving it.

On p. 422 the Author had quoted the definition of equivalent single wheel load, but there were some who contended that the effects of loads transmitted through multiple-wheel undercarriages to pavements could not be expressed in terms of equivalent single wheel load. Perhaps in theory there was some basis for their contention, but the Paper had demonstrated that for practical purposes such an equivalent could be assessed, and that the margin of error was not such as to invalidate the reliability of the results. Further, Mr Mann believed that it was an essential aid in airfield design to be able to express design criteria in terms of a single load, and he thought that to attempt to design pavements on the basis of a series of applied loads with interaction occurring presented, in the present state of knowledge, a most formidable and, to say the least, tedious task to resolve satisfactorily.

The Author had very clearly shown that for thin pavements the equivalent single wheel load was the load transmitted by each individual wheel of the undercarriage, but that as the depth of construction increased interaction developed until a point was reached when there was an indisputable equivalent single wheel loading value. The Paper was valuable because it demonstrated how aircraft with those multiple-wheel undercarriage assemblies would produce different stresses in pavements according to the type and depth of pavement construction, however inconvenient that might be to the designer of the aircraft. It also brought out most forcibly that it was not the all-up weight of the aircraft which mattered, but the means adopted to transmit the load to the ground. It directed attention to the large number of uncertainties in airfield design.

The civil engineer's aim was to ensure that the subgrade strength, whether it was natural or enhanced by mechanical or chemical stabilization, was used to the greatest advantage; that the best was got out of the subgrade; and that no more than was necessary was spent to provide a wearing surface to make good the deficiency between the applied load and the bearing strength. Assistance was obtained from soil mechanics, but there were so many variable factors which could not be pre-determined that designers might well be excused if they were averse from taking chances, because they would certainly be blamed if there was failure. There was, of course, strong pressure to reduce the cost of construction of airfields, but much of that cost was due, not to engineers having built too thick pavements to deal with multi-wheel undercarriages, but largely to the length which was now required for runways, and indeed the increased width also.

If the Paper did nothing else—and Mr Mann thought that it did a great deal more—it should dispel any lingering idea that airfield pavement design was an exact science, akin to other branches of what might be called civil engineering, such as structural steel design and so on. It was unreasonable to expect engineers deliberately to reduce design strength below that which they believed was necessary to carry the pavement without failure. Considering the unknowns in the problem of the condition of the subgrade (over which the designer might have no control, in years to come) and the variables (shown in the Paper) which might occur in the stressing owing to changes in the aircraft which might use the runway, the little factor of safety which engineers introduced—and which some people thought was excessive—was not really so excessive as it might appear at first sight.

The Author had shown that those problems were still under investigation, and it was hoped to have exact answers, in time, which would make possible a further reduction in the cost of pavement construction.

Professor S. C. Redshaw expressed the opinion that the Author had done a very good service by summarizing the present state of knowledge with regard to pavements, but thought that, just as modern earth pressure theory had been evolved from theoretical methods such as those of Rankine and Coulomb, so it was about time to turn away from Boussinesq and start from fundamental principles on runways. The soil which had to be dealt with did not necessarily agree, of course, with Boussinesq's ideal conception, and it was desirable to get down to basic principles again. That need was underlined by the fact that $\cos^2\alpha$ was altered to $\cos^3\alpha$ because that fitted the curves better.

The use of an equivalent single wheel load seemed to find great favour, but possibly it was an over-simplification, for which runway designers were not yet ready. The Author had stated that each aircraft had a different equivalent single wheel load for each airfield, and Professor Redshaw thought that the conception of the equivalent single wheel load might be masking some other effects. The C.B.R. test was, he thought, a stopgap measure which was, from its very empirical nature, open to criticism.

Mr Scott Hall had referred to the static load being the worst case, and it had always been understood to be so, but what was the effect of accumulated compaction due to repeated landings? The aircraft normally landed at one end or other of the runways; could the Author provide any information about any bad effects from compaction over a length of time, and on whether the effect of vibration from stationary aircraft with running engines had a bad effect on the runways themselves?

There were numerous theories about runways, but he did not think that considerable progress would be made until some practical measurements could be taken. Before that could be done, he thought that a really accurate pressure cell which would measure the pressures in the subgrade of various runway constructions would be necessary. At Birmingham University great efforts were being devoted to the pure matter of instrumentation, in an attempt to produce a suitable pressure cell for that purpose.

The Author had mentioned model-tests, and they should be very valuable if the right technique could be used, although some tests up to date had been disappointing; it was naturally an extremely difficult subject. Probably the only way would be to approach the problem with an idealized model, and then inevitably it would be necessary to proceed to a large-scale model of a practical runway. One of the troubles with models, of course, was that after a few loadings the model had been spoilt, and it was an extremely extravagant way of testing. Did the Author endorse the view that that was the best approach, or did he think that good results could be obtained only by full-scale measurements?

Mr D. H. Little observed that there were two approaches to a difficult problem: it could either be admitted to be difficult and an approximate answer accepted, or it was possible to resort to higher mathematics. The design of flexible pavements seemed to be largely based on the C.B.R. method. In that method, as was well known, a small sample of earth was taken and a plunger forced into it, and, because it worked in California, it was hoped that it would work in Somerset. It was an excellent example of the approximate approach, but for rigid pavements most people resorted to higher mathematics.

The Paper seemed to be a combination of both approaches. The Author had said at the end of it that the various Air Ministry practical tests had led to no definite conclusions, and it might be assumed that they had neither proved nor disproved the Westergaard theory. That, however, did not worry the Author, who took the theory, as Mr Little would have done—there being nothing better to take. The Author could not work out the coner case, it being too difficult—Mr Little did not think that Westergaard had worked it out either—and so he had taken the centre case, which again was what Mr Little would have done. But on that very "approximate" foundation the Author had then built up a rather terrifying mathematical edifice.

Reading similar Papers, and Papers such as the Road Research Laboratory had published recently, in which were given about twenty different methods, none of which were trusted, of designing runways or pavements, it would seem that the time had come to have an approximate or what might be termed a "C.B.R. equivalent method" for rigid pavements. At any rate it was necessary to have something during the next few years, while the Author was doing his laboratory work.

Mr Little illustrated the method he had adopted by considering a 40,000-lb. wheel load. That sounded a big load, but if referred to as 20 tons it became less impressive. If carried on a 12-inch-by-12-inch column, it resulted in a base pressure of 300 lb. per square inch, which also sounded disturbing when thought of in terms of tire pressure. But that, in turn, could be expressed as 20 tons per square foot, and it made the problem easier from the start if those less disturbing units were used. Ignoring the corner-load question, because that was too difficult, and considering only the centre condition, the problem resolved itself into the designing of a building foundation, and, looking up a standard structural book, it would be found that the bending moment in a square base was $\frac{W}{8}$. No matter how big the slab might be, it was $\frac{W}{8}$, provided the whole of the slab was brought into play. If the slab was made 3 feet 3 inches square, giving an area of 10 square feet, there would be a subgrade pressure of 2 tons per square foot, and if the slab was 10 feet by 10 feet—100 square feet—the pressure on the ground would be 4 cwt per square foot. If it was 20 feet by 20 feet the pressure would be 1 cwt per square foot. If the ground was very poor the whole of the slab would be needed; if the ground was good and the slab was still 20 feet by 20 feet, only a portion of it would operate and that portion would act as a built-in beam, and the bending moment might be $\frac{W}{16}$ instead of $\frac{W}{8}$. Somewhere in between that range was the right answer.

There were three ways of eliminating the corner load: to reinforce heavily; to have load-transfer devices; or to have twin slabs. The constructions which he had had most to do with had been twin slabs. In the case of two slabs acting together it was reasonable to suppose that the bottom one would give some additional support, and the moment would be between $\frac{W}{10}$ and $\frac{W}{20}$, depending upon the type of ground.

That method of approach had the following advantages. It was simple to understand and to use. It took into account the strength of the subgrade and the size of the panels. He had not seen the latter mentioned before, but he felt that the size of the panel was a very important factor, especially where the range might be 10 feet by 10 feet to 20 feet by 20 feet. It dismissed tire pressures as being of no importance, and, with slabs 20 feet by 20 feet, common sense suggested that it did not matter whether a central 20-ton load sat on an area 1 foot by 1 foot or 2 feet by 2 feet. When dealing with a multiple-wheel load, it would be seen from p. 432 that the twin wheels there sat on an area only about 4 feet by 1 foot 6 inches, and, frankly, in the middle of a slab 20 feet by 20 feet he did not think that it would make any difference at all whether there was one wheel or two wheels; the distribution over an area 4 feet by 1 foot 6 inches in relation to 20 feet by 20 feet was neither here nor there. In fact, the Author seemed to come to the same conclusion in his example, because his double load was 49,000 lb. and the equivalent single load was 39,000 lb. In Mr

Little's opinion no pavement calculations were yet exact enough to distinguish between 49,000 lb. and 39,000 lb. The four-wheel case dealt with on p. 435 would be spread over an area equivalent to 7 feet by 4 feet. That was not a very big spread in relation to a slab 20 feet by 20 feet; by Mr Little's method it was possible to take it into account, but the reduction in the bending moment would not be very much.

He did not offer the method as an exact one. He did not know as much about the work as did the Author, but the method had been tried out. Designs had been built based on it in Scotland, in two or three parts of England, and in Northern Ireland, and Mr Little hoped that it might work even in California.

Mr E. H. Lewis-Dale observed that he did not regard the Paper merely as welcome, or valuable, or worth reading; in his view something of the kind was absolutely essential. He did not think that it mattered much how an airfield pavement was designed, unless a great many were involved, but it did matter a lot that it should be possible to say, after designing the pavement, what it would carry. It also mattered a lot that other people should be able to say what their airfields would carry. Aircraft designers and air operators would probably agree with him there. Great Britain, and all the member States of the International Civil Aviation Organization, were committed to publish the strengths of airports internationally.

Up to the present the situation, as he saw it, had been that an aircraft could be designed or specified by the operator and built to the formula given and published, and then some country would have second thoughts about the strength of their airfield pavement and would decide that the aircraft could not land there at all. So as not to offend any country he would not quote names, but it was stated in a current publication that an asphalt runway of unspecified thickness could take an all-up weight of 330,690 lb.—no number of wheels, no tire pressure, no spread of load, nor anything else. A very famous airport, not in Great Britain, had a concrete runway, of unspecified thickness, which could take a gross load of 300,000 lb.; no details of the spread of the load, no tire pressures or other data were given. Probably one day, when somebody had designed an aircraft within that figure, with very small wheels with intense tire pressures, and it showed every sign of breaking up the runway, it would be decided that that form of classification was not good enough.

It would be a very good idea to offer the Author's method to the world, to be adopted unless investigation proved another method to be better. It was an easy method; anybody could learn to use the Tables in a quarter of an hour, as Mr Lewis-Dale had done.

Mr J. H. Siddons referred to the diagram of vertical pressures under a flexible pavement shown in *Figs 2*, and asked whether there was any experimental proof that interaction of the wheels could and did cause higher pressures in the centroid between the wheels than under the wheels themselves. It would be noticed, he said, that in the figure the highest pressure shown was between the wheels and not under the wheels. He suggested that where the wheels were applied simultaneously, as would happen with a normal aircraft undercarriage, the pressures vertically under the wheels must always be greater than anywhere else in the same horizontal plane, and he explained that suggestion by considering the vertical plane midway between the wheels. Each side of that plane was subject to equal downward drags by the spread of the load through the basecourse, and that was therefore, he considered, a plane of zero shear. In consequence, there should be no transfer of load from either wheel across that plane, and therefore no superimposition of one wheel load on the other.

That did not mean that the pressures were the same as those imposed by only one of the wheels. Since one side of the cone of pressure distribution under each wheel had been cut off by the plane of zero shear, the area subjected to pressure at any particular depth was therefore reduced, and the pressures occurring in the remaining area should increase correspondingly. The maximum pressures occurring vertically under the wheels would therefore be greater than that developed under one of the wheels acting independently. He suggested that those maximum pressures might be somewhat less than the maximum pressure developed at the centroid of the area according to *Fig. 2*, and until experimental evidence was available, it might be reasonable to take the increase in pressure as proportional to the decrease in the bearing area.

He believed that the Road Research Laboratory had an experiment in hand to verify the spread of pressure under a single load, and he wondered whether it could be readily extended to investigate the effect of this multi-wheel loading. The immediate requirement from a practical point of view was to ascertain the maximum pressure that was found under wheel loading in terms of the total load applied, and, as a secondary consideration, where that maximum pressure occurred. Lastly, it was necessary to know what the total distribution of pressure would be ; but the important thing was to know what was the maximum pressure which might be expected.

Mr J. A. Robertson observed that there was one point in the excellent Paper which the Author had presented which he did not think had been sufficiently emphasized, either by the Author himself or by subsequent speakers, and that was the fundamental weakness of the idea that it was possible to express the strength of pavement by one number. He thought that a very big error arose from that one idea, and it was time that a new system of classification was agreed by the civil engineers and the aircraft designers, so that the degree of concentration of the load as well as the load itself, whether on 1, 2, 3, 4, 8, or 20 wheels, could be given. Then a curve or a small formula could be used, in which the total load and the distribution could both be given perhaps not precise, but adequate, weight.

Mr J. F. Varcoe asked why a designer decided to construct a rigid pavement or a flexible pavement, and asked the Author to enlarge on that. Was there or was there not a tendency today to go away from rigid pavements to a flexible design ? Mr L. D. Hicks, Chief Soils Engineer of the North Carolina State Highway, a Public Works Commission, had a theory that the term " flexible construction " should not be used, but that pavements were " fligid." In other words, they were neither rigid nor flexible, but something in between. That would make the Author's problem more difficult, but perhaps he would consider it.

Mr J.A. Dawson questioned whether the Author was right in suggesting that there was no interaction of load in the case of a twin-wheel assembly on a thin pavement, whereas there was with a thick pavement. The interaction must take place somewhere, even though it occurred below the pavement itself, and account must be taken of that in designing the pavement, because if there was a greater pressure under the centroid rather than under the wheel it was necessary to be sure that the subgrade was capable of taking that load, or failure would take place. If the point of greatest pressure was under the centroid and below the thickness of the pavement it seemed to him that there was no need for any basecourse and that under those conditions a wearing coat only was required.

⁂ **Mr S. P. Banerjee** remarked that the Paper was of particular interest to him, because he had recently had the opportunity of investigating the design of pre-stressed concrete end slabs for runways, on behalf of his firm.

He noticed that in the evolution of the design curves the Author had based his method on Dr Westergaard's " centre-case " formula, given in 1926, for the design of roadway pavements. That had been revised in 1933 with an additional term, since the original formula gave excessively high stresses.

The formula developed for the purpose of runway pavement design had been given by Dr Westergaard in 1939 as a further modification to the 1933 formula and took the following form :—

$$\sigma_i = \frac{0\cdot275P}{h^2}\left[1+\mu\right]\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right) - 54\cdot54\left(\frac{l}{L}\right)^2 z + 0\cdot582\left(\frac{kb^4}{Eh^3}\right)^{\frac{1}{2}}\right]$$

as expressed with the notations given in the Road Research Technical Paper No. 20, pp. 32 and 33. The significance of the modification has been discussed by Mr J. N. McFeeters.[1]

To compare results, however, Mr Banerjee had analysed in detail some such slabs by the " Beams on Elastic Foundation " theory, which was based on the well-known differential equation $EI\frac{d^4y}{dx^4} = -ky$, EI being the flexural rigidity of the pavement slab. Considering two-way distribution of pressure, that method indicated that Westergaard's values were slightly conservative. That might be accounted for by the customary

---

⁂ This contribution was submitted in writing—SEC. I.C.E.

[1] " Concrete Runways." Airport Paper No. 2. I.C.E., 1946.

employment of the minimum possible values for the co-efficients $\frac{l}{L}$ and z in the formula, the actual values of which depended so much upon the relative stiffnesses of the slab and base in a particular case.

Whilst Mr Banerjee appreciated the method presented in the Paper for rapid design of runway pavements with multiple wheel loads, he was unable to understand why the Author chose to make use of the 1926 formula which was obsolete in all respects for the purpose. Mr. Banerjee added that the 1939 formula might, for convenience of application, be re-expressed in the following form, in line with equation (5) of the Paper, when $\mu = 0{\cdot}15$:

$$\sigma_i = 0{\cdot}316\frac{P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 0{\cdot}531 + 0{\cdot}191\left(\frac{l}{b}\right)^{-2}\right]$$

wherein the $\frac{l}{b}$-ratio still remained the only variable.

Mr Banerjee pointed out that the constant 1·0693 given in equation (5) should read 0·967 for $\mu = 0{\cdot}15$.

The Author, in reply, welcomed the suggestions that had been put forward by the contributors to the discussion.

Mr Scott Hall had mentioned the liaison which had been established between the civil engineer and the aircraft designer. This liaison was most valuable, and the introduction of the multiple-wheel undercarriage was one result of it.

Only the static case was considered because the relationship established should hold for both the static and kinetic cases. There was not, as yet, much evidence regarding moving loads and, until such evidence became available, it was necessary to rely on static conditions. The problem had to be reduced to narrow limits, and this was also the reason for assuming that the subgrade was the critical level for failure in a flexible pavement. It was specifically stated in the Paper that " this assumption will normally hold in a carefully designed pavement where a margin of excess strength is included in each of the base-course layers." If there was any doubt about that, the particular case would have to be investigated in detail to find whether there was any plane of weakness, other than the plane of the subgrade. The case of wearing-coat failure mentioned by Mr Scott Hall, would have occurred equally with a single-wheel or multiple-wheel aircraft.

Regarding Load Classification Numbers, the Author had endeavoured to stress that, whilst any airfield pavement had a fixed L.C.N., a particular multiple-wheel aircraft had different L.C.N. values for varying types of construction. The value for any particular airfield could be assessed readily when the construction details were known. The stiffness of a rigid pavement was the measure of its strength and L.C.N., but the L.C.N. of a multiple-wheel aircraft varied with the pavement thickness.

Referring to plate-bearing tests, the Author said that, in the field, it was not possible to carry out single-plate and multiple-plate tests under the identical conditions required to give a comparison. In order to assess an equivalent single wheel load for two or four wheels by field experiment, it was necessary to carry out a plate-bearing test with a multiple-plate group, and compare the resulting stressing conditions in the slab with that occurring under a single plate. It might be necessary to wait a year before identical conditions of temperature, moisture, and all the other variables, which would affect the reaction of the slab, occurred for the performance of the second test. The desired result was a ratio which was assumed to hold for any set of conditions. Those factors did not mean that the assessment of runway Load Classification Numbers by plate-bearing tests was suspect, since due regard was paid to the conditions obtaining when the tests were performed when L.C.N. was estimated. The L.C.N. was the loading combination which would produce failure when the atmospheric and other conditions were such as to reduce the strength of the pavement to the minimum.

Mr Scott Hall's reference to P.S.P. was interesting. The behaviour of P.S.P. was still not fully understood, and intensive investigational work was required to develop a valid theory for its action in relieving loading effects on a subgrade.

The Author thought that the ultimate aircraft, which would not require a runway, would be very welcome in relieving the civil engineer of one of his most difficult problems, but it seemed that it would be many years before this ideal would be achieved, and it would probably bring other problems in its train.

Mr Mann had emphasized the difficult position of the airfield designer. It was very easy to criticize the latter. If an airfield pavement failed, he was accused of under-design. He might even be accused of under-design if the pavement failed under an aircraft twice the weight of that for which it was designed. If it did not fail, he was accused of over-design and insufficient regard to economy, and such accusations were very difficult to refute. Again, it should be remembered that a rigid concrete pavement gained in strength, so that its load-bearing capacity after 2 years would be almost twice that at 28 days, but a designer could not prevent traffic using the airfield 2 or 3 months after the last batch of concrete had been place.

The Author, referring to Professor Redshaw's remarks, said that, so far as he knew, Boussinesq's original theories of earth pressures were still the civil engineer's main stand-by in soil mechanics. Variation of the "concentration factor," as suggested by Fröhlich, seemed a reasonable expedient to cover the case of varying moduli of elasticity of the soil layers. Burmister's analysis[1] gave another possible approach, but that method was too complicated for direct application to the particular problem discussed in the Paper. It might be found that the use of different methods would produce very little variation in the equivalent single wheel ratio.

---

[1] D. M. Burmister, "The Theory of Stresses and Displacements in Layered Systems and Applications to the Design of Airport Runways." Proc. 23rd Ann. Mtg High Res. Bd, Wash. D.C., vol. 23, 1943.

Extensive loading experiments on an ideal uniform soil had been carried out by the Corps of Engineers, U.S. Army, and the Report[1] did, to some extent, justify the Boussinesq formula for such a soil. The great difficulty of the pressure cell, as mentioned by Professor Redshaw, was shown in those tests, and it seemed that that problem could never be solved completely. It was not possible to design a pressure cell of very small size, and, so long as it had any depth, it was liable to produce an odd stressing condition around it which would affect the readings. The Author hoped that the work in hand at Birmingham University would produce an answer to that problem.

The Author considered that, since tests had to be carried out under controlled conditions for reasons already given, full-size models would be impracticable and, for satisfactory results, scale-model tests would be required. The scale of the models should, however, be as large as possible.

Mr Little's proposals were interesting, but the Author thought that he was carrying simplification too far. A limit had to be placed somewhere, and the methods in the Paper were thought to represent the maximum degree of simplification which could be permitted; they could be used at once, without waiting for the result of the laboratory experiments. Mr Little's "foundation slab" theory ignored two factors, firstly, the uneven distribution of reaction under the slab due to variable deflexion, and secondly, the variation in reaction conditions arising as the load moved from the centre towards the edge or corner of the slab. The latter affected design more than it affected the equivalent single wheel load ratio, but the former did have to be considered, and undoubtedly the wheel spacing was very important.

Mr Little had wondered whether considerable accuracy was really required but, when the airfield pavement designer was being asked to justify every fraction of an inch in thickness and every penny of expenditure, he would find it rather difficult to say, "I thought that the equivalent single wheel load was 49,000 lb., but it could just as well be 39,000 lb."

The Author agreed with Mr Lewis-Dale that there was a need for a clearer international system of classification for civil airfields. It was absolutely pointless to say that an airfield could take a wheel load of, say, 30,000 lb., unless the associated time pressure was also quoted.

Mr Siddons had not been convinced that the total vertical pressure at depth in a flexible pavement was a maximum at the centre of the group of wheels, and not under each of the wheels. If the wheels of a twin-wheel undercarriage were actually touching, then obviously, at depth, the maximum pressure would occur centrally under the group. If the wheels were then separated, the maximum stress would remain on the centroid of the pair until, at a certain spacing, the degree of interaction would decrease to such an extent that the maximum vertical pressure would occur directly under the individual wheels. The Corps of Engineers, U.S. Army, experiments referred to, showed clearly that the maximum pressure at depth did, in fact, occur at the centroid of the group of wheels.

---

[1] Corps of Engineers, U.S. Army, "Investigations of Pressures and Deflexions for Flexible Pavements, Report No. 1, Homogeneous Clayey-Silt Test Section." Technical Memorandum No. 3-323, U.S. Wtwys. Exp. Stn., Vicksburg, Miss., March 1951.

Mr Robertson's suggestion of factors to give the degree of concentration of load under 1, 2, 4, 8, or 20 wheels would hardly be practicable as they could take no account of variation in spacing and pavement characteristics.

Mr Varcoe's query regarding the choice of a flexible or rigid pavement was, the Author said, rather outside the scope of the Paper. It had been dealt with recently by Dr B. H. Knight.[1] On a very poor subgrade such as a fatty clay, with a C.B.R. value of about 3 per cent, a very heavy wheel load might require a depth of flexible construction of 5 or 6 feet, whereas, for a rigid pavement, only about 16 inches of concrete would be necessary. On a very good subgrade, such as gravel with a C.B.R. value of about 30 per cent, a flexible surfacing 9 inches thick would suffice for the same wheel load, but 10 inches or more of concrete would still be required if cracking was to be prevented. There was a subgrade strength above which it would normally be more economical to use a flexible pavement. Other factors had to be considered as well, such as the possible adverse effects of jet aircraft on flexible surfacing which might demand the use of a rigid pavement. Economic considerations and the materials supply position could influence the decision considerably.

In reply to Mr Dawson, the Author said it was true that interaction commenced very near the surface of a flexible pavement. The reference to grass-surfaced airfields possibly gave the best answer to the query. If the bearing value of the soil was constant to a considerable depth, it could be assumed that the rate of decrease of vertical pressure on successive horizontal planes in the depth of the soil would be greater than the rate of increase of interaction, and therefore, the maximum vertical pressure, either directly under the individual wheels, or under the centroid of the group, would always be less than that under one wheel near the surface. A soil capable of carrying the wheel load in its natural state would have a very high C.B.R. value, perhaps 50 per cent and would require a surfacing as a wearing coat only.

Mr Banerjee had asked why Westergaard's later formulae were not used. The Author said that the original formula was much more fully developed, and that facilitated the preparation of the equivalent single wheel load curves. He could not agree that the original formula was obsolete:

---

[1] B. H. Knight, "Engineering Factors Influencing the Adoption of Rigid and Flexible Pavements." Airport Paper No. 16, Instn Civ. Engrs, 1951.

some authorities considered that the later formulae underestimated the stressing conditions. Since the centre case was being analysed with a view to application on the critical corner case, it was reasonable to use a formula which would overestimate rather than underestimate the stresses. However, Mr Banerjee's suggestion would be followed up to determine whether the later formulae gave a significant difference to the results.

An arithmetical check had been made on the constant 1·0693 in equation (5) for $\mu = 0{\cdot}15$, and the Author could not agree that it was incorrect.

Correspondence on the foregoing Paper is now closed, and no further contributions other than those already received at the Institution, may be accepted.——Sec. I.C.E.

ROAD ENGINEERING DIVISION MEETING

4 March, 1952

Brigadier A. C. HUGHES, C.B.E., T.D., B.Sc., M.I.C.E., Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Road Paper No. 37

"Recent Developments in Highway Bridge Design in Hampshire"

by

Edwin William Henry Gifford, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

The Paper describes the various types of construction used in recent years, discusses their relative merits, and compares pre-stressed concrete with traditional methods, demonstrating its advantages for small- and medium-span highway bridges.

A brief description of bridges constructed in reinforced concrete and in concrete-filled steel troughing is given, followed by a more detailed account of pre-stressed bridges already constructed, as follows:

(*a*) Four road bridges of 34 feet span. These are of slabs built up with pre-cast longitudinal beams transversely pre-stressed in place.
(*b*) Six footbridges varying in span from 34 feet to 43 feet built up from 4-foot-long pre-cast hollow units assembled into beams and longitudinally pre-stressed on site.
(*c*) Two 70-foot-span footbridges with side girders made from pre-cast sections and carrying pre-tensioned slab decking.
(*d*) A large number of very small span (15 feet–20 feet) footbridges built with pre-tensioned planking of various types.

Further, two bridges under construction or projected are described. These are a 25-foot-span road bridge in a busy main street where the use of pre-cast post-tensioned decking is being used principally to reduce site time, and a 70-foot-span road bridge pre-cast in small sections and assembled on site by pre-stressing.

Details are given of a full-scale test on a 34-foot-span beam of the type described in (*a*), together with some remarks on design and load factors.

INTRODUCTION

Two extreme views on pre-stressed concrete are prevalent in Great Britain. Some engineers have been so impressed by its novel qualities that they have failed to recognize its relationship to traditional methods and so treat it with excessive caution, whilst others, by thinking of it merely as a further development of reinforced concrete, fail to make full use of its unique properties.

By means of a brief account of traditional methods and a more detailed description of several pre-stressed concrete works and designs, this Paper attempts to show how this new material is related to the older forms of construction and describes some of the features peculiar to its use.

Although the distinction may sometimes be a little difficult to make, it is possible to describe any form of bridge construction as either pre-fabricated or in-situ, and it is interesting to note that each new kind of construction has, when first introduced, often been of the alternate group to the one preceding it.

The earliest form of timber construction, that of felling a standing tree directly across a stream is certainly pre-fabricated, and this description can also be applied to the more developed forms where all the members are cut to size and shape before being fastened together on site. Timber still has, for small-span bridges, the advantage of very rapid construction.

Brick and masonry followed as more permanent substitutes, but with the disadvantage of slow construction with its resulting prolonged obstruction to traffic—a factor of some considerable importance to the present-day highway engineer.

Iron and steel provided materials capable of extensive pre-fabrication such as shop cutting, drilling, and riveting together of sections and sometimes of complete spans. Once again, rapid construction was possible but more maintenance was required than previously.

The claim made for reinforced concrete when it was introduced was that it reduced maintenance and decreased the number of skilled men required for construction. But bridge works reverted to the in-situ category and bridge reconstruction became a slow process. Some of the earlier work was not as durable as had been expected and spalled concrete with exposed reinforcement is seen much too frequently.

Pre-stressed concrete combines the advantages of the speedy construction that comes from pre-fabrication with the low maintenance requirement previously associated only with in-situ work. Indeed, pre-stressed concrete members should achieve a greater resistance to corrosion and weathering than any other material except perhaps the very best of dry-jointed masonry such as that used in the Pont du Gard, which is still in good condition after nearly 2,000 years.

The problem often facing the highway engineer, when a bridge is to be reconstructed, is that he must balance economy in his design with the cost in inconvenience to the public while the works are in progress. The economical structure should have low maintenance-cost and a long life, and for this reason reinforced concrete has been used extensively and, when properly designed, has lasted well. But it has the disadvantage of slow construction and is unsuitable for pre-fabrication because of its great weight. A typical example of reinforced-concrete construction is Water

Bridge, Hurstbourne Priors (*Fig. 1*, facing p. 466), built in 1950. This has a simple slab of 28 feet span, resting on brick-faced mass-concrete abutments.

For many years various forms of steel troughing have been used in Hampshire, particularly when rapid reconstruction has been required, as, for instance, when it has not been convenient to rebuild a bridge in two halves to allow uninterrupted traffic. The disadvantage of steel troughing is the difficulty of waterproofing the deck and the uncertainty about its condition after many years of service. A further limitation is that the simplicity of construction may be lost if the troughs are distorted in handling so that the holes have to be re-drilled or reamered on site.

An example of this type of construction is Wade Bridge, near Romsey (*Fig. 2*). It was built in 1947, and has three 18-foot spans, each resting on reinforced-concrete piled trestles.

It is evident that road bridges should be quick to build, require the minimum of maintenance, and have a low first cost. Speed requires prefabrication, low maintenance means good steel protection, and low first cost is achieved by economy in materials and labour. These points are all met by pre-stressed concrete :—

1. The comparatively light weight of pre-stressed concrete allows medium spans to be prefabricated in complete beams, launched into place and transversely pre-stressed. Where the spans are too large for this, the beams may be cast in small sections, longitudinally pre-stressed on site and then united by transverse pre-stressing.
2. The steel is well covered by very-high-quality concrete, which, in the majority of designs, is always in compression. This is quite the reverse of the condition in reinforced concrete or steel joists in concrete, where the bottom concrete is always in tension and permanently cracked. Nevertheless, well designed reinforced-concrete bridges show no sign of corrosion after nearly 50 years of service (a sizable bridge built in 1904 in Hampshire is still in good condition) so that the very much better conditions attained in pre-stressed concrete are a satisfactory indication of its potential resistance to weathering.
3. The saving in quantity of materials has been frequently described, being approximately 50 per cent concrete and 70 per cent steel. This is offset by the greater unit cost of the higher quality required, but overall savings in cost of 10–20 per cent over other forms are usual.

FACTORS OF SAFETY

There are at present two distinct approaches to pre-stressed concrete design :—

(*a*) The design of the structure for the service load to produce certain limited stresses.

(*b*) The design of the structure to fail at a given multiple of the service load.

In either case it is necessary to decide upon a factor of safety. It has been suggested by some authorities that since a stress relationship of 3 is used in steel and reinforced concrete between working stresses and ultimate stresses, then the same value of 3 should be used as the load factor against failure for pre-stressed concrete. This reasoning is based on the assumptions that, first, the failing load is of fundamental importance in the design and, secondly, that a stress factor of safety of 3 is the same as a load factor of 3. It is, however, suggested that if the service load is properly assessed then the engineer's consideration should be given to the maximum load that the bridge will carry without permanent damage and that the load factor should be determined with respect to this "crippling load" and not to the failing load.

The load factor for steel and reinforced-concrete bridges for this condition (which is at yield point for the tension steel) is approximately 2¼ and may be less—the exact value depending on the shape of the section and the proportion of dead to live load—whereas the stress factor and so-called factor of safety is 3. But experience has shown that with bridges properly designed on this basis crippling loads do not occur, so that it would be reasonable to design pre-stressed concrete bridges also with a load factor against permanent damage of 2¼.

Now one of the characteristics of pre-stressed concrete members is that they may be loaded almost to failure before suffering any permanent damage or serious non-elastic deflexions and that a load factor against crippling of 2¼ will mean failure at about 2½. It is evident therefore that a factor of 3 against failure will mean a crippling factor considerably above that in use at present for reinforced concrete, which, it is contended, would be unnecessary and uneconomical.

A second characteristic of pre-stressed concrete which further protects it from damage from overloading is that of high elastic deflexion at the upper end of the load/deflexion curve, although service-load deflexions are less than for corresponding beams in mild steel or reinforced concrete. These high deflexions give ample warning before the non-elastic range is reached, whereas such deformations in steel or reinforced concrete would mean that permanent damage had already taken place.

A further consideration in the design is the load at which cracking will occur. One of the great advantages of pre-stressed concrete construction over reinforced concrete is that under conditions of dead load only (which cover the greater period of a bridge's life) the concrete in the bottom of the beam is in compression and that under working loads the tensile stress is either zero or considerably less than that of tensile cracking. This means that a pre-stressed member is much more resistant to corrosion than reinforced concrete, and this advantage should be maintained by making the cracking load sufficiently in excess of the working load to make its occurrence very rare indeed. It should be emphasized that any cracks that do occur are immediately sealed after the load is removed, but will re-open at lower loadings than that required to crack the concrete initially, but if the beam is properly designed and the cables are well bonded, these cracks will not re-appear at working load but about half-way between this and the first cracking load.

It is not therefore essential that the cracking range should be as high as that suggested for permanent damage and a value of 1½ is probably reasonable. If there is zero stress in the bottom fibre under service load, then a factor of 1½ against cracking is roughly in accordance with a factor of 2¼ against crippling.

The exact values of the load factors suggested depend on the engineer's opinion of the validity of the design service loading used and on the probability of its being exceeded. For routes frequently used by abnormal loads, such as the heaviest military tanks, it will probably be advisable to use a higher value than 2¼ times Ministry of Transport Standard Loading. However, it is better design practice to assess the maximum service load as accurately as possible than to allow for its inaccuracies by increasing the load factor. The Ministry of Transport have introduced a Heavy Loading for certain routes and it is suggested that this could be applied, with the load factors of 1½ and 2¼ previously described, to bridges where there is a probability of the frequent occurrence of loads greater than those provided for by the Standard Loading.

A test has been made on one of the 34-foot-span road-bridge beams to check these design assumptions and this is reported in the Appendix.

THE USE OF PRE-STRESSED CONCRETE IN HAMPSHIRE

The first application of the pre-stressing technique in Hampshire was made in 1948 when a number of the 30-foot-span Ministry of Transport emergency-bridging beams were used in the reconstruction of Lindford Bridge, Bordon. These are hollow pre-tensioned box sections designed by the late Dr Mauhtner and were manufactured in 1940. The beams were used in substitution for a normal reinforced-concrete deck and proved very simple to place. Since the beams had been intended for use on bridges with any angle of skew, no provision for transverse pre-stressing had been made and point loads are spread by a 4-inch concrete screed containing two layers of road mesh reinforcement. This bridge has now been in service for 4 years (11 years after the beams were made) on a road used by a considerable volume of Army tank traffic and is perfectly satisfactory.

The first County Council design was for Water Bridge—a 28-foot-span structure. The official design employed pre-tensioned T-beams but contractors were invited to submit alternative designs. However, the tenders received were high in comparison with reinforced concrete, which was therefore used instead. The reason for these high prices appeared to be that contractors who were at that time sufficiently experienced in pre-stressed concrete to submit a low price for the beams were unable to produce a competitive figure for the site works, whereas local contractors were over-cautious in pricing for pre-stressing. Also, it has since been realized that the shape of the beam was unnecessarily complex for such a small span.

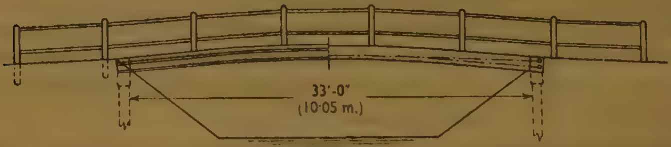
In the next scheme to be prepared, therefore, the design was simpler, and the manufacture of the beams was put out to separate tender so that the main contractor had only to "receive and fix." In that way low prices were obtained and this method of tendering is now used for all pre-stressed bridges in the County.

Description of Bridges Completed or under Construction

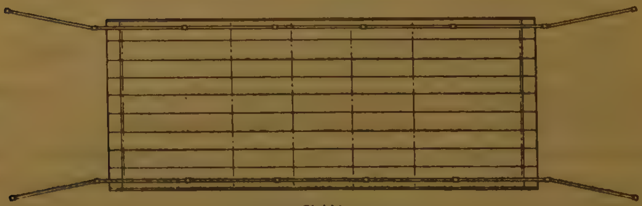
Road Bridges

Four bridges of 34 feet span have been built, and a 25-foot span is under construction.

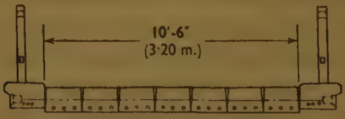
Figs 3



ELEVATION
SHOWING LONGITUDINAL CABLES



PLAN
SHOWING TRANSVERSE CABLES

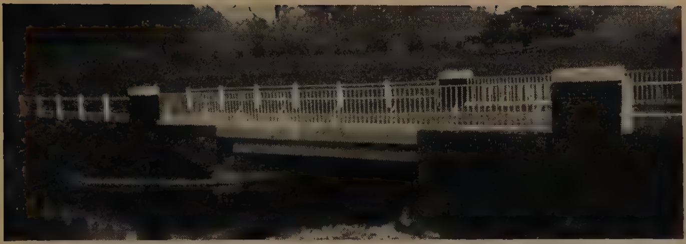


CROSS-SECTION AT MID-SPAN
Scale: ⅛ inch = 1 foot

RHINEFIELD BRIDGE

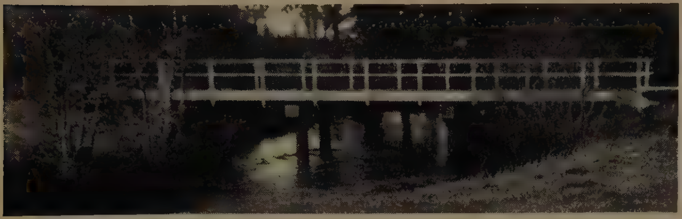
(*a*) 34-*foot span* (*Figs 3*).—The first to be built was Rhinefield Bridge, in the Rhinefield Inclosure of the New Forest, which carries a 10-foot-6-inch-wide road over a 20-foot-wide stream. To reduce site construction

Fig. 1



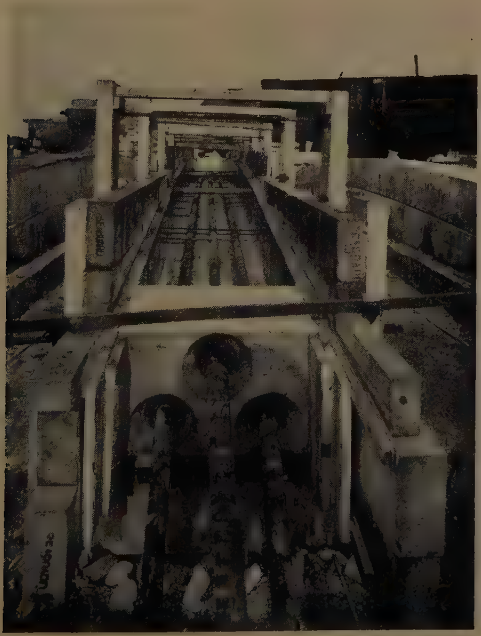
Water Bridge, Hurstbourne Priors. A Typical Reinforced-Concrete Slab Bridge of 28 feet Span

Fig. 2



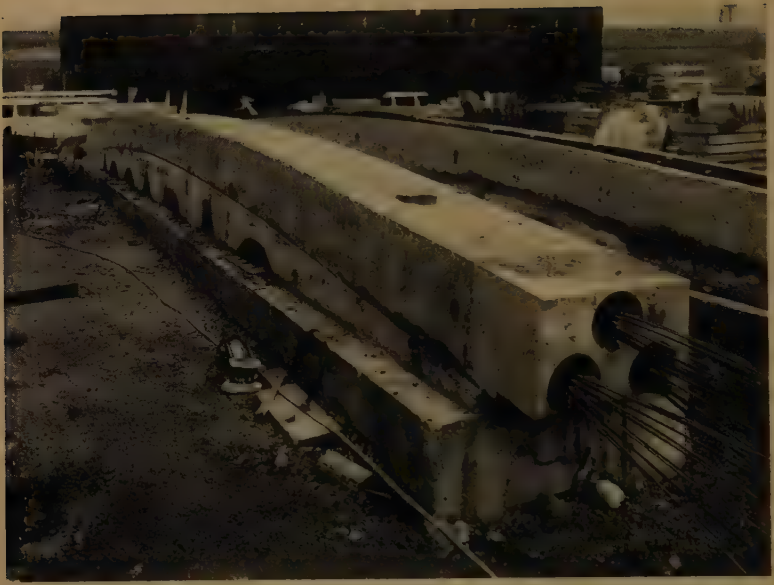
Wade Bridge, near Romsey. An Example of Steel Troughing Construction

Fig. 4



End View of Beam Mould, showing the Pre-Cast End Anchorage Block in Position and the Method of Securing the "Ductube"

Fig. 5



Deck Beam after Pre-stressing. The ¾-inch Separation Between the Beam and its Bed can be seen in Comparison with the Unstressed Beam Alongside

Fig. 6

Unloading Beams with a 5-Ton Mobile Coles Crane time, as much of the bridge as reasonably possible was pre-cast. These bridges were therefore designed with beams of rectangular section 12 inches deep and 18 inches wide, the contract for manufacture being obtained by the Liverpool Artificial Stone Company at Bermondsey, London. The abutments were simple piled trestles with reinforced-concrete capping beams on 12-inch-by-12-inch pre-cast concrete piles, and were placed at the top of the river banks so that water-work was avoided.

It is considered that beams of up to at least 40 feet long are cheapest when pre-cast under factory conditions and transported to the site, and that for spans from 25 feet to at least 35 feet the post-tensioned pre-cast slab section is more economical than in-situ slabs, I-beams or composite construction.

To produce a pleasing elevation, the beams for this first bridge were designed with a 12-inch rise and were cast on four concrete beds shaped to this curve, with two sets of timber side-moulds and vibrated with "Sinex" mould vibrators. The concrete was of Thames ballast of $\frac{3}{4}$-inch maximum size with a 4 : 1 aggregate/cement ratio and 0·42 water/cement ratio, which gave a workable concrete with a cube crushing strength for air curing of 5,000 lb. per square inch in 4 to 5 days and a 28-day strength of about 8,000 lb. per square inch. The end anchorage blocks, containing the cones and the secondary reinforcement (which was a 6-inch-diameter helix of mild-steel wire round the cone and a double mesh of $\frac{1}{4}$-inch high-tensile wire at 2-inch centres in front of the cones), were steam cured to produce a cube crushing strength of 6,500 lb. per square inch at 48 hours. This strength could, in time, have been attained by normal air curing, but steam curing was used to speed up the start of manufacture. The Author considers that a high strength in the anchorage block at the time of stressing is very important and although in these beams the cones had a minimum cover of only 1 inch of concrete in no case did this spall. The anchorage blocks were used as stop-ends to the moulds, the sides being clamped to them with sash cramps, and the ducts formed with "Ductube" located with transverse steel pins which were withdrawn a few hours after the concrete was cast (*Fig. 4*). Six-inch concrete test cubes were made from each beam and were tested before the beam was stressed, a cube strength of 5,000 lb. per square inch being required for this operation; this strength was normally attained 4 to 5 days after casting. Each beam is stressed with three cables each of twelve 0·2-inch wires and each initially tensioned to 69,000 lb. from one end only. After allowing for the usual losses this is equivalent to a final load of 58,500 lb. (72 tons per square inch).

The method of cable tensioning from one end only is to drive in the male cone at the fixed end by a few blows of a 4-lb. hammer on a wooden drift, to mark the wires at an arbitrary distance of about 6 inches from the female cone face, and then to jack the wires from the other end. When the calculated extension has been reached on the jack, the amount that the wires have pulled in at the fixed end is measured (usually about $\frac{1}{4}$ to

$\frac{3}{8}$ inch) and this is added to the theoretical extension at the jacking end, plus the usual $\frac{3}{16}$-inch allowance for slip in the jacking-end cone which occurs when load is transferred from jack to anchorage.

The upward deflexion at pre-stressing was approximately $\frac{3}{4}$ inch, and it is important to note that to obtain equal upward deflexion the beams should be stressed at the same age (*Fig. 5*). The tests carried out on one of these typical beams are described in the Appendix.

The parapet beams were manufactured in a similar way, using the same casting beds but with modified mould-sides. The top and outer sides of these beams were bush-hammered to expose the aggregate and the surface finish produced is well shown in *Fig. 8*.

The beams had a maximum weight of $3\frac{1}{2}$ tons; they were transported three at a time on a long trailer and were easily off-loaded and handled by a 5-ton Coles crane. The crane slings were shackled directly to the end of the beam (*Fig. 6*) because it was found that a spreader beam was not necessary.

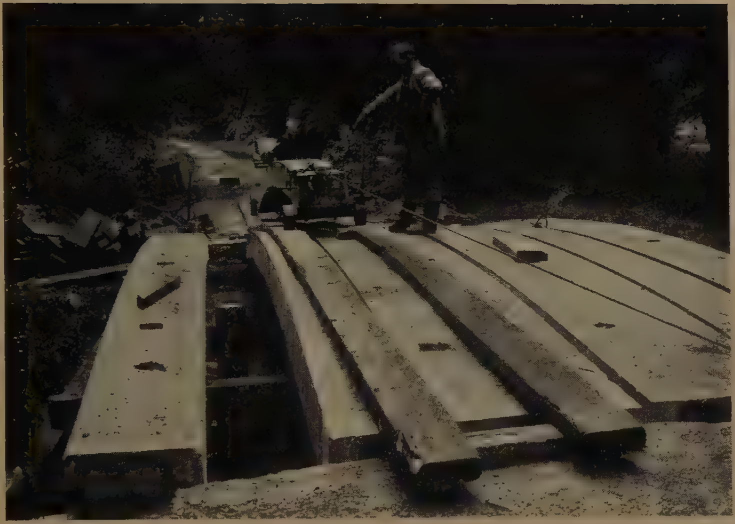
The beams were placed on small wheels and three carriageway beams were rolled across the existing bridge, which was then demolished, and the remaining beams were winched across over the first three and jacked and rolled sideways into place (*Fig. 7*). The longitudinal joints between the beams were caulked with a dry 3 : 1 sand/cement mortar and the deck was then transversely pre-stressed with six cables, the tensioning starting at one abutment and working towards the other (*Fig. 8*).

Although no noticeable lateral movement of the beams occurred during transverse pre-stressing, this was no doubt because of the small width of 14 feet, but on wider bridges provision should be made for the transverse strain to take place without undue restraint. On the fourth 35-foot-span bridge constructed, which has a width of 24 feet, this was done by placing a layer of waterproof paper between the top of the abutment trestle and the cement mortar upon which the outer beams were bedded.

The second 35-foot-span bridge was built over the Moors River at Hurn, near Christchurch, and also carried a road only 10 feet 6 inches wide, but this time the full clear span of 33 feet was used by placing the beams on sheet-piled abutments. The beams were identical with those at Rhinefield and although the cost per super foot of road over waterway was slightly less than at Rhinefield it is now realized that it would have been better to have used longer beams with piled trestle abutments at the top of the banks; the extra cost of the beams would have been offset by the saving on the sheet piles and the site time would have been considerably reduced. The Contractor for these bridges was Nettons Limited of Christchurch.

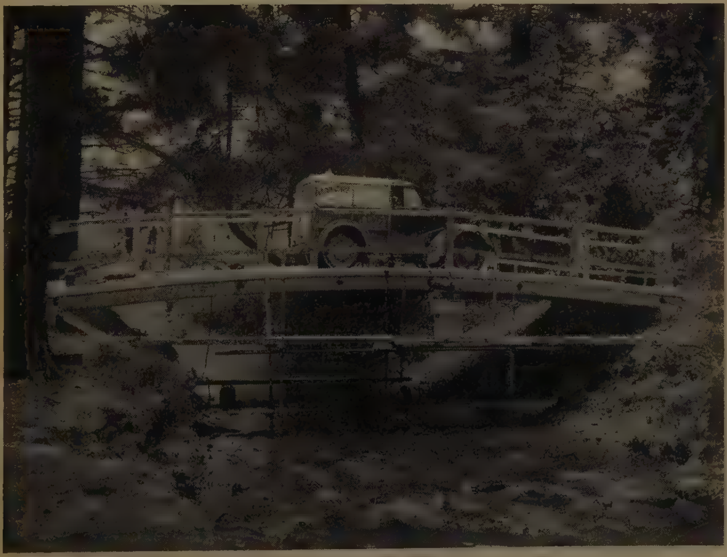
The third and fourth bridges (Bridgefoot, near Plaitford, and Cadnam Green) were built with beams with a rise of only 4 inches, since the traffic speed on these roads was much greater than that at the first two bridges. The parapet beams were also made with greater depth to serve either as

Fig. 7



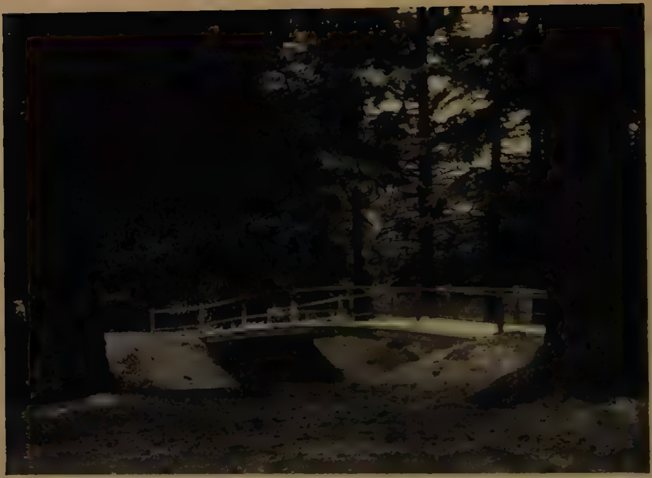
Rhinefield Bridge. Hauling Beams into Position

Fig. 8



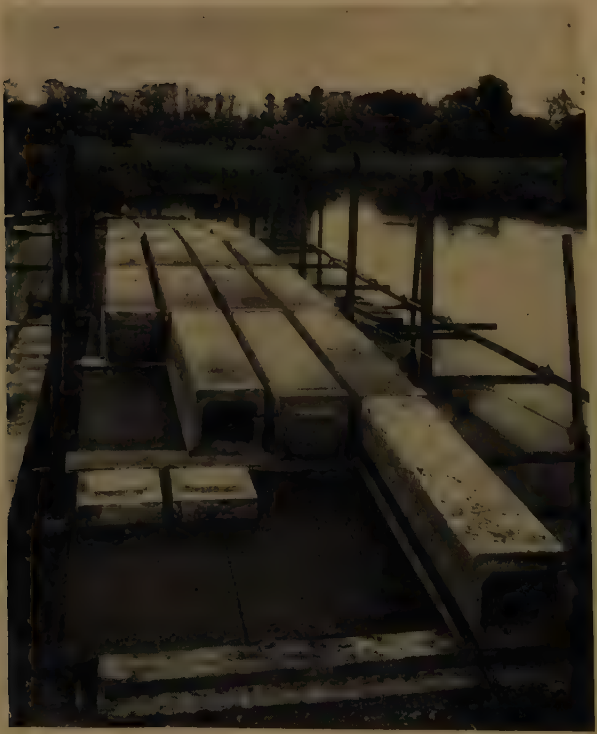
Rhinefield Bridge: Transverse Pre-Stressing

Fig. 9



RHINEFIELD BRIDGE, NEW FOREST

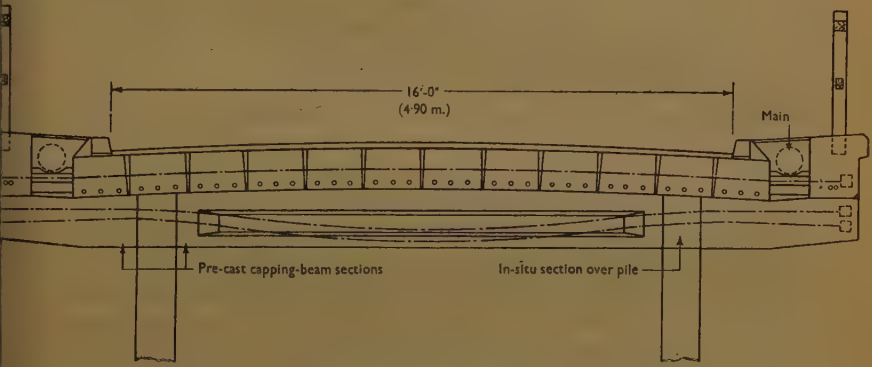
Fig. 14



HIGH FOOTBRIDGE, NEAR WINCHESTER. PRE-CAST UNITS ON TEMPORARY STAGING kerbs (as at Bridgefoot bridge) or as backing to the footpath (as at Cadnam Green bridge).

Bridgefoot was in all other respects similar to Rhinefield, but the Cadnam Green bridge, although built with similar beams to Bridgefoot and on a piled trestle abutment, has two interesting variations (*Fig. 10*). The first is that provision is made for mains by separating the parapet and kerb beam by spacers, through which pass the transverse pre-stressing cables. These spacers also assist in supporting longitudinal pre-stressed planks, which carry the gravel with which the duct is filled. The spacers are of sufficient stiffness to transfer a portion of the live load to the parapet beam. The second feature of this bridge is that the capping beam to the abutment trestle was pre-cast in three sections and post-tensioned. After the two piles of the trestle were driven, the reinforcement was stripped down to the required level and the pre-cast sections of the capping beam were set in place. The pre-stressing cables were next threaded through and the exposed section over the piles was wrapped with paper to prevent bonding with the short in-situ section over the pile head. This was cast next, using aluminous cement, and the capping beam could have been stressed as early as 24 hours afterwards had it been necessary. This in-situ section allowed adjustments to be made for inaccuracy in driving and ensured a good bond with the pile reinforcement.

Fig. 10



CADNAM GREEN BRIDGE

The contractor for this second pair of bridges (J. J. Udalls (Building) Company, Limited, of Southampton) unloaded the beams in a different manner to that described for the first pair. Two sets of sheer-legs were placed 35 feet apart in line with the new bridge, one set being just behind the abutment. The vehicle carrying the beams was driven under these and the beams were lifted up while the lorry was driven out again. The beam was lowered on to its wheels and hauled directly across the bridge while the lorry backed in for the next beam to be unloaded.

The cost of these bridges ranges from £4 10*s*. to £5 per square foot of road over waterway.

(*b*) 25-*foot-span road bridge* (Figs 11, Plate 1).—This small bridge spans the River Anton in Andover and it is proposed to reconstruct it in a method similar to that used for the 35-foot-span bridges, although 25 feet is near to the smallest economical span for this method. However, since it is necessary to build this bridge in the short period between the drop in water level in Spring and the beginning of summer road-traffic and to have the minimum of obstruction on this busy road, the same pre-cast section has been used. The deck will be constructed in two sections, the transverse pre-stressing being arranged in two operations in such a manner that the second half is properly tied to the first. The footpath is made of pre-tensioned planks, some of which are fastened to both the kerb beam and the parapet beam and also serve to support services and mains which can be exposed by removal of the remaining slabs.

The parapet beam will be faced with white concrete, ¾-inch thick, cast with the ordinary concrete and designed to give approximately the same strength.

Footbridges

Hampshire County Council is responsible for the maintenance of approximately five hundred footbridges on rural paths, and the number is increasing now that the Access to the Countryside Act has made the highway authority responsible for all footpaths on public rights of way.

The existing bridges are mainly of timber and since they are often in remote places their maintenance is expensive, involving frequent inspections, painting, and minor repairs.

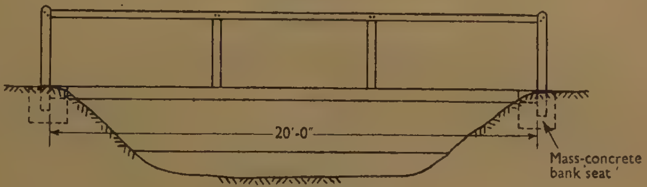
Reconstruction in timber is costly and the new structure is often short-lived, and for these reasons more permanent materials are to be preferred. Steel and reinforced concrete have both been used but are not entirely satisfactory. The steel bridge requires painting and is rarely of pleasing appearance, whereas reinforced concrete is expensive in remote sites and looks rather heavy. Pre-cast reinforced-concrete units have been used but are suitable only for short spans or sites with vehicular access.

Several types of design in pre-cast pre-stressed concrete have been developed in Hampshire for varying ranges of span and these so far appear to be the best solution.

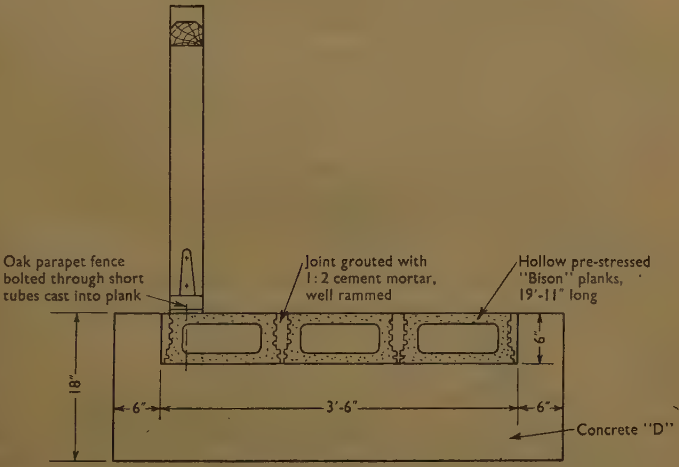
Small spans, 15 *to* 20 *feet* (*Fig. 12*).—The first units used were pre-tensioned solid slabs, 4½ inches deep, 12 inches wide, and 15 feet long. These were very economical and a 2-foot-wide footbridge with single galvanized tube handrail and concrete bank-seats could be built with them for £20.

Twenty bridges were built with these, but now the lighter "Bison" flooring unit is being used. Bison units are hollow and have been adapted for footbridge use by casting-in sections of gas-pipe for handrail fastenings. These "planks" are designed to carry a load of 60 lb. per square foot, and have a depth of 4¼ inches for 14 feet span and of 6 inches for 19 feet span, and weigh 5 and 8 cwt respectively. The hollow planks are placed side by side on the previously prepared abutments and the joints between them are grouted-up with a stiff mortar. The handrails are then fixed and the bridge is surfaced. These are cheaper than the solid unit and their much lighter weight simplifies handling so that the 14-foot-span bridge, 2 feet wide, can be built for £17 10*s*., and 19-foot-span, 2 feet wide for £30. These prices are less than for bridges of medium-quality timber, when this is available, and of course the pre-stressed concrete is much more durable.

Figs 12



ELEVATION
Scale: ⅛ inch = 1 foot


Oak parapet fence bolted through short tubes cast into plank
Joint grouted with 1:2 cement mortar, well rammed
Hollow pre-stressed "Bison" planks, 19′-11″ long
18″
6″
6″
3′-6″
6″
Concrete "D"

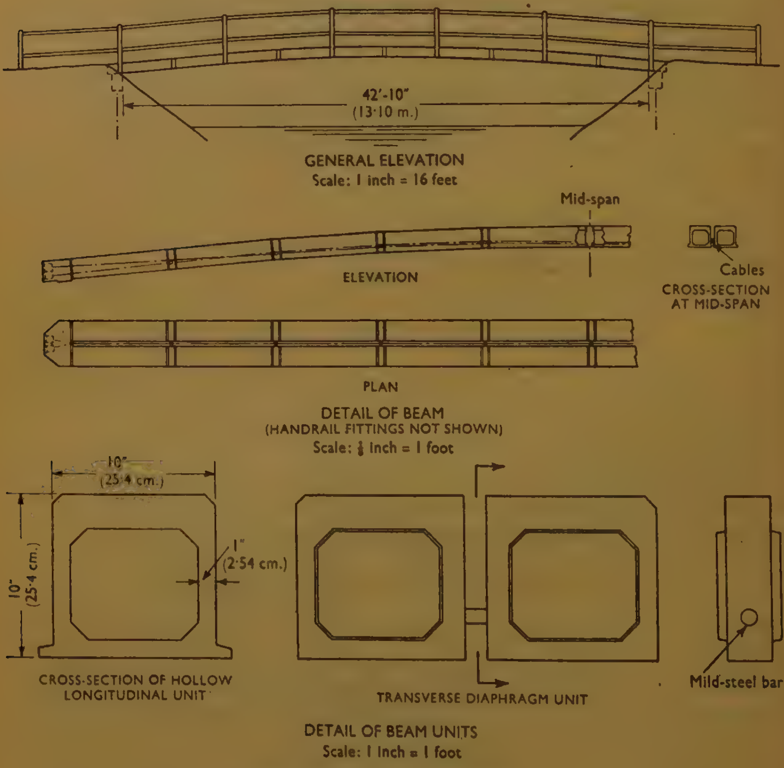
CROSS-SECTION AT MID-SPAN
Scale: ½ inch = 1 foot

20-Foot-Span Footbridge

Medium span, 34 to 43 feet (*Figs 13*).—These, and the longer spans, are designed for a uniformly distributed load of 50 lb. per square foot.

Many footbridge sites have such difficult access that it would be impracticable to use beams of this span pre-cast in one length, and so a segmental beam was designed. The hollow rectangular units, 10 inches by 10 inches by 4 feet overall, are arranged in pairs with the cables running between them, their ends bearing on diaphragms which also serve to locate the cables, which are anchored in end blocks placed across the end pair of hollow units. The walls and bottom of the hollow unit are only 1 inch thick and were made with ½-inch-maximum-size aggregate without any difficulty. The concrete had a cube strength of 5,000 lb. per square inch at 7 days, and a maximum working stress of 2,100 lb. per square inch at the time of initial pre-stress of the 43-foot-span bridge.

Figs 13



43-foot-Span Footbridge

The units were factory pre-cast and were hauled as near as possible to the bridges by road and from there were carried by hand or barrow to the site, each unit being designed as not greater than a two-man load.

The sections were placed on light temporary staging shaped to the upward curve of the beam ; this curve, as for the 35-foot-span road bridges, was introduced for appearance only (*Fig. 14*).

The joints between the longitudinals and the diaphragms were made by injecting a grout that adhered to the sides of the joint. The required consistency and strength were obtained with 0·6 part of fine sand to 1 part of "417" Portland cement, the water/cement ratio being 0·42, and to give smoothness a small quantity of wetting agent was added to the water. Air at a pressure of 40 lb. per square inch from a small portable compressor, forced the cement grout from a pressure pot of the paint-spray type along a rubber hose and out through a curved nozzle into the joint (*Fig. 15*). The grout had a 6-inch-cube crushing strength of 4,200 lb. per square inch at 48 hours, although in fact most of the joints were stressed at ages greater than this. After each joint had been made, the adjacent longitudinal unit was forced against its corresponding diaphragm unit and the squeezed-out mortar was struck off with a trowel.

The cables were threaded between the units, their positions being fixed by the mild-steel bar between the diaphragms and stressed from one end only, as described for the 34-foot-span road-bridges. The 34-foot-span footbridges have one cable per 22-inch-wide beam, tensioned to 60,000 lb. and the 43-foot-span bridges have two cables per beam each tensioned to 40,000 lb. When stressing was completed the longitudinal joints between the units were filled with fine concrete, the oak fence was fastened on with bronze studs and bushes, and the bridge was surfaced (*Fig. 16*).

Udalls (Building) Company, Limited, were the contractors for these bridges.

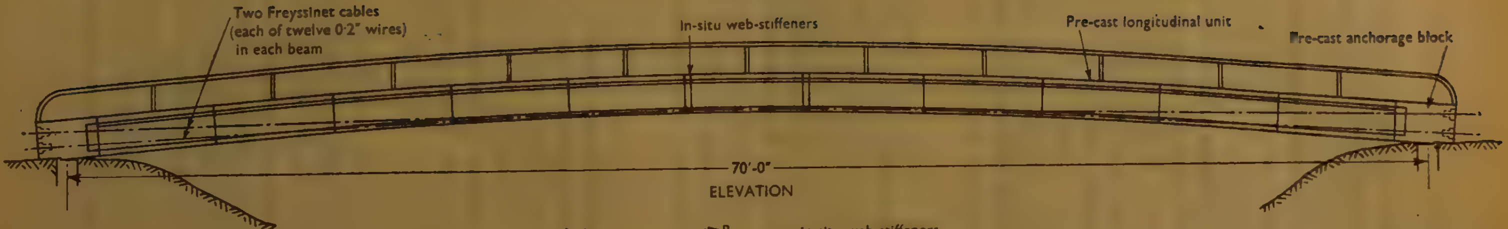
Costs of this type of construction worked out at between 17*s*. 6*d*. and 20*s*. per square foot for the deck alone, which is comparable with steel and considerably cheaper than reinforced concrete for bridges in remote sites.

Longer spans 50 *to* 70 *feet* (*Figs 17*).—One 70-foot-span footbridge has been built by Nettons, Limited, and a second is under construction. These have two longitudinal I-section girders, 23 inches deep, pre-cast in 6-foot-long sections, the first bridge being pre-stressed with two 12-wire Freyssinet cables in each beam and the second with two 1-inch Macalloy bars.

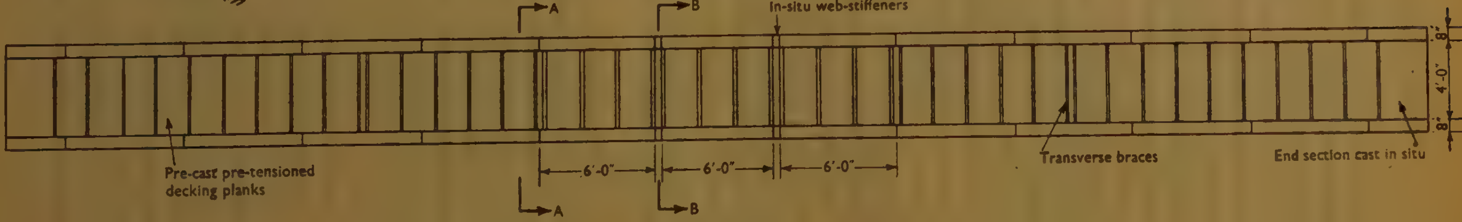
The beams are transversely stiffened by pre-cast concrete braces secured to the lower part of the girder web by ½-inch-diameter Macalloy bars. The deck is formed by transverse concrete planks, each 1½ inch thick and 20 inches wide, pre-stressed with eight pairs of 10-gauge wires. These are stressed to 2,100 lb. and give a uniform pre-stress of 1,000 lb. per square inch, providing a load factor of 2 on the design load of two 150-lb. point loads on each 20-inch-wide plank, to allow for accidental overloading by cattle.

The beam segments were factory-pre-cast and taken near to the site by road and then by a small two-wheeled bogie for the remaining distance.

Figs 17 (a)



ELEVATION

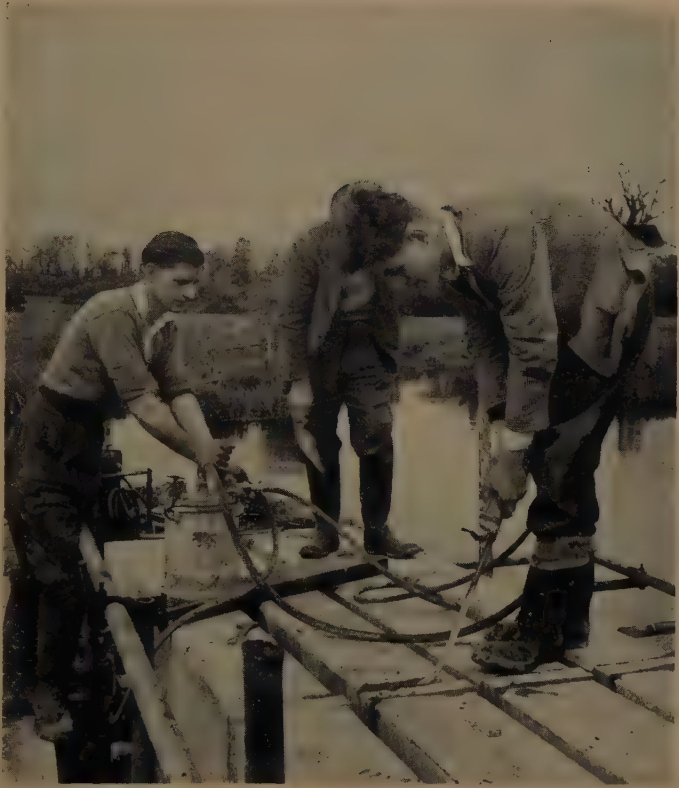


PLAN
(PARAPET OMITTED)

Scale: 1 inch = 10 feet

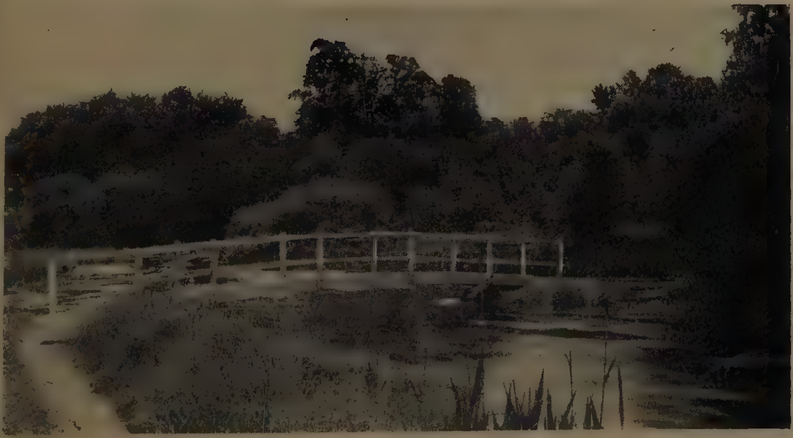
70-FOOT-SPAN FOOTBRIDGE

Fig. 15



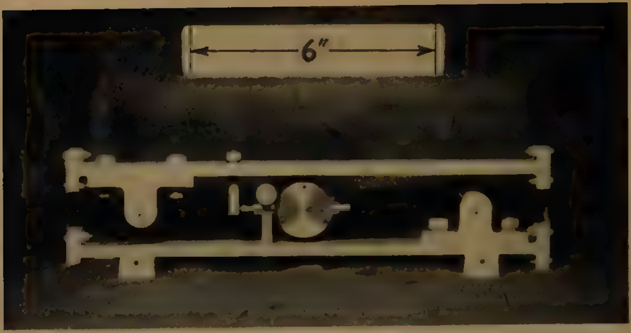
Grouting Joints between Units

Fig. 16



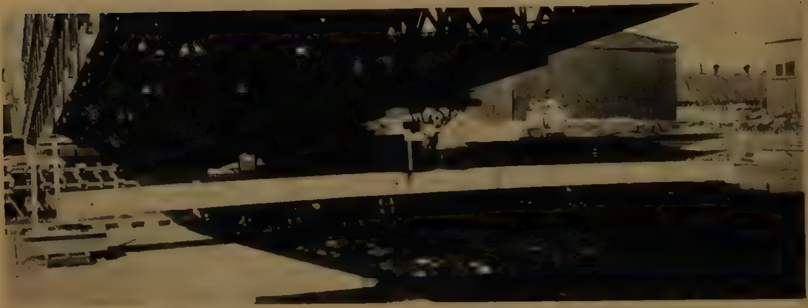
High Footbridge Completed

Fig. 20



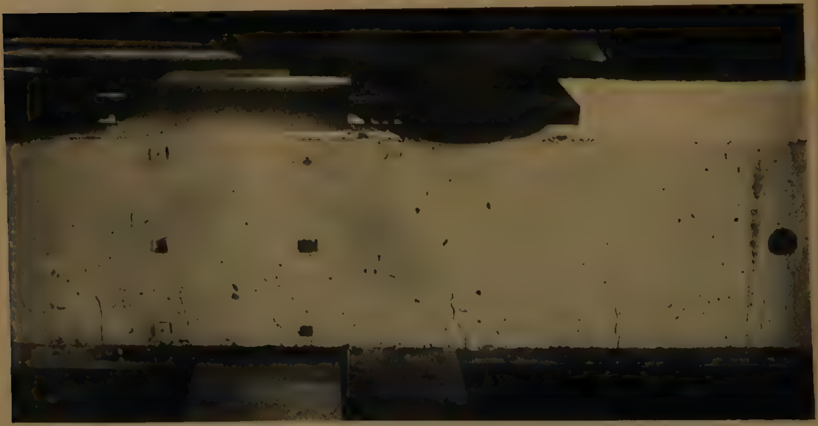
DISMOUNTABLE STRAIN GAUGE

Fig. 22



TEST BEAM IN POSITION, SHOWING LEVEL-STAFFS AND LOADING POINT

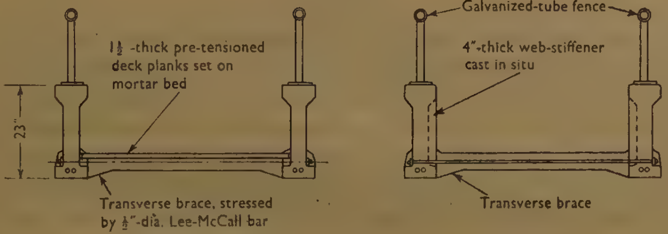
Fig. 28



CRACKS AT MID-SPAN AT 6-TON LOAD. APART FROM THE TWO UNDER ONE LOADING POINT (WHERE FAILURE ULTIMATELY OCCURRED) THE CRACKS ARE UNIFORM IN DEPTH AND DISTRIBUTION

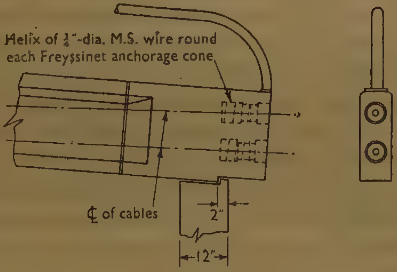
The units were set up on the temporary staging, and the cables drawn through with a leading wire and sheathed at the joints, which were then caulked with cement mortar. This was made with two parts of sand to one of cement with the minimum of water to give cohesion, and driven into place with a caulking tool. The cables were tensioned from one end only and grouted in the usual manner.

Figs 17 (b)


Galvanized-tube fence
1½-thick pre-tensioned deck planks set on mortar bed
23"
Transverse brace, stressed by ½"-dia. Lee-McCall bar
4"-thick web-stiffener cast in situ
Transverse brace

SECTION AA

SECTION BB


Helix of ¼"-dia. M.S. wire round each Freyssinet anchorage cone
℄ of cables
2"
12"

DETAIL OF ANCHORAGE

Scale: ¼ inch = 1 foot

70-FOOT-SPAN FOOTBRIDGE (SEE *Figs 17* (*a*))

FUTURE WORKS

Canal Bridge (Figs 18, Plate 2)

This design has been produced as a solution to the problem of providing a bridge across a 35-foot gap, where the required vertical line of sight combined with the statutory headroom over the canal limits the construction depth to 12 inches. Because of the possibility of Army tank traffic and because of the width of the bridge, the 34-foot span beams previously described would not be strong enough, and it was decided to design the bridge with three spans, the central section to be suspended. Theoretically it would be possible to reduce the construction depth to less than 12 inches, but this would make arrangement of the suspended-span bearings very difficult.

Because of the large angle of skew, the longitudinal component of the transverse pre-stressing force would probably be sufficient to cause adjacent beams to slide upon each other. To overcome this, recesses will be formed in the sides of the beams at each transverse cable and these will be filled with concrete after the beams have been placed, the longitudinal force being taken by these inserts acting in shear.

Test Bridge, Stockbridge. 70-foot span. (*Figs 19*)

This bridge will carry road No. A.30 across the Test at Stockbridge and the design is based on the experience gained from the segmental footbridges.

The whole bridge deck will be pre-cast in small sections, none of which will be heavier than 17 cwt, so that it can easily be assembled with a small mobile crane. The main-beam units, which are cast complete with diaphragms, are 6 feet long and will be lined up on site and assembled by pre-stressing. This may be done either on the bank and the first beams hauled across either the existing bridge or a temporary staging and then rolled sideways, or on a scaffold across the river wide enough for one beam, and again moved sideways into place. When all the beams for the first half of the bridge are in place on the abutments, the bottom flange and diaphragm joints will be caulked with cement mortar and the bottom transverse cables placed in position. Then the top-flange units will be fixed and the top transverse cables stressed; these are arranged so that the top flanges can be stressed separately for each half of the bridge, in a similar way to that described for Andover Town Bridge. The bottom cables will not be stressed until the whole deck is completed, so until this has been done wheel loads will be distributed over several beams by a temporary roadway of timber railway sleepers laid in sand on top of the completed first half.

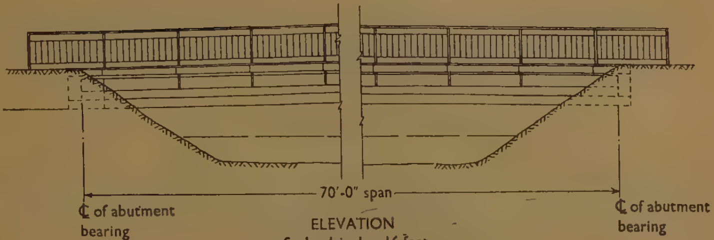
The parapet beam will also have additional top-flange sections fixed after the beam has been placed and held by the transverse cables. The bottom transverse cables will stop at the kerb beam so that the space between this beam and the parapet will be available for mains and services, which could also be threaded through between the carriageway beams.

The reasons for selection of this design are as follows :—

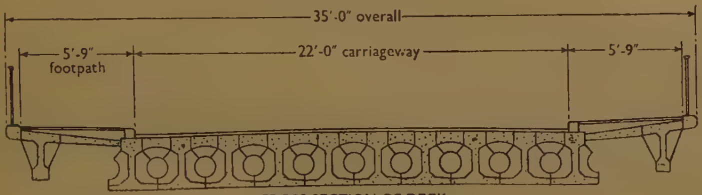
(1) For spans of up to about 80 feet the hollow slab is probably most satisfactory as it gives a shallow construction depth and good transverse stiffness. For spans of more than 80 feet it may be more economical to use T-beams, braced with transverse diaphragms.

(2) The small number of beams required would not justify site casting and since a 70-foot beam would be difficult to transport, it is considered more convenient to cast the beams in short lengths and assemble them on site.

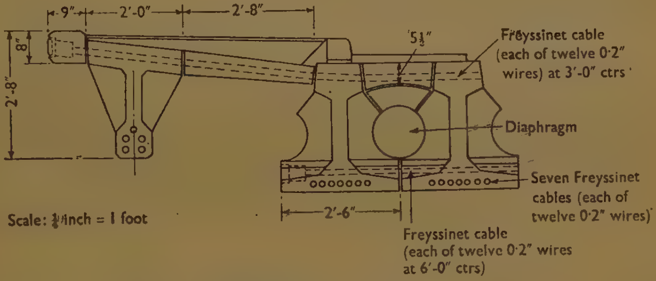
Figs 19



ELEVATION
Scale: 1 inch = 16 feet

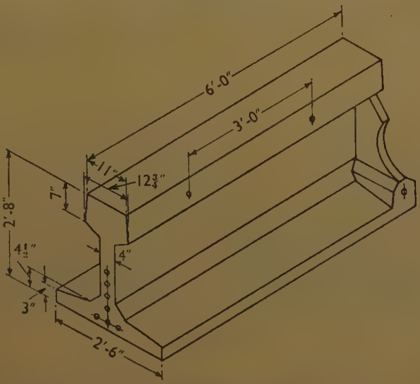


CROSS-SECTION OF DECK.
Scale: 1 inch = 10 feet


9″
2′-0″
2′-8″
8″
2′-8″
5¼″
Freyssinet cable (each of twelve 0·2″ wires) at 3′-0″ ctrs
Diaphragm
Seven Freyssinet cables (each of twelve 0·2″ wires)
2′-6″
Freyssinet cable (each of twelve 0·2″ wires at 6′-0″ ctrs)

Scale: ¼ inch = 1 foot

MID-SPAN CROSS-SECTION OF FOOTPATH



TYPICAL CARRIAGEWAY BEAM UNIT

RECONSTRUCTION OF TEST BRIDGE NO. 219, STOCKBRIDGE TRUNK ROAD NO. 8 (A.30)

(3) The omission of the top flanges reduces the beam weight from approximately 12 tons to 9 tons and simplifies the caulking of the bottom flange joints and the construction of diaphragms.

ACKNOWLEDGEMENTS

In acknowledging his thanks to the County Surveyor of Hampshire, Brigadier A. C. Hughes, B.Sc., M.I.C.E., for permission to publish the Paper, the Author wishes to state that the views expressed are his own and do not necessarily conform to those of the County Surveyor or the Minister of Transport.

Thanks are due also to the following: Mr A. J. Harris, B.Sc., A.M.I.C.E., of the Pre-stressed Concrete Company, Limited, for general assistance and advice during the preparation of the designs; to Professor A. D. Ross of King's College, London, for advice on testing; to the staff of the Military Experimental Establishment at Christchurch for assistance in carrying out the beam test; to the Cement and Concrete Association, to Mr E. V. Cox, and to the Liverpool Artificial Stone Company, Limited, for permission to publish photographs.

The Paper is accompanied by seventeen photographs and fourteen sheets of drawings and diagrams, from which the half-tone page plates, Plates 1 and 2, and the Figures in the text have been prepared; and by the following Appendix.

APPENDIX

REPORT OF TEST OF 34-FOOT-SPAN ROAD-BRIDGE BEAM

PURPOSE

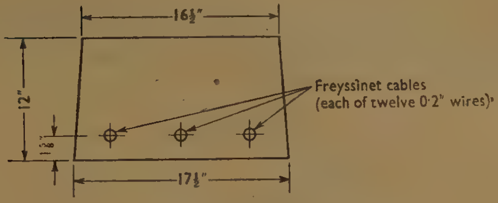
This test was made as a check on design methods and assumptions which were briefly as follows:—

(1) That pre-stressed concrete conforms with elastic theory within the range of working stresses.
(2) That a load factor of 1·5 against cracking is adequate protection against cracking at working loads even after the beam has already been cracked.
(3) That repeated overloads do not have a detrimental effect on the beam.
(4) That a pre-stressed concrete beam can be loaded to within 90 per cent of failure without permanent damage and that a load factor of 2·25 against this condition is reasonable protection.

APPARATUS

(1) *Beam*

The beam used was a standard pre-stressed concrete road-bridge beam of 34 feet effective span and having the following characteristics.



MID-SPAN CROSS-SECTION

$I = 410.$ M.R. = 700,000 lb. in.

Equivalent mid-span point load = 3·06 tons, producing stresses of 1,700 lb. per square inch.

Initial pre-stressing force of 69,000 lb. on each cable which, combined with dead load bending moment will produce stresses of + 2,145 and — 215 lb. per square inch. After relaxations this will produce stresses of + 1,750 and — 110 lb. per square inch.

It was at first intended to test a special beam without holes for transverse pre-stressing cables, but for realism a production beam was finally used.

(2) *Strain Measurement*

Strain readings were taken with a dismountable mechanical gauge of the Macklow-Smith type (*Fig. 20*, facing p. 475). This measured the change in distance between pairs of mild steel plates glued with "Durafix" to the surface of the beam. These plates were fixed 20 centimetres apart and the dial gauge on the instrument could be read to 2×10^{-3} millimetre so that strains of 1×10^{-5} could be detected.

The first readings with this instrument were inaccurate, probably because the tapered pins on the instrument were not pressed sufficiently firmly into the small holes in the gauge plates.

The readings for the pre-stressing of a second beam were much better, although some of these readings were still irregular, but this time the operator was probably affected by the rain in which he was working.

On the final loading test of this beam, however, the readings were completely satisfactory and remarkably consistent.

(3) *Loading*

Each end of the beam rested on stools, one of which was fitted with rollers to permit horizontal movement.

The load was applied at mid-span by a hydraulic jack working against an overhead frame, and was measured by a calibrated ring (*Figs 21* and *22*).

The beam was loaded as follows :—

(1) By 1-ton stages up to cracking load of 5 tons.
(2) " " down to zero.
(3) Items (1) and (2) repeated 5 times.
(4) By ½-ton stages up to 7 tons.
(5) " " down to zero.
(6) Items (4) and (5) repeated once.
(7) To failure.

Deflexions

Deflexions were measured to 0·02 inch by a level sighting on to scales set up at mid-span and at each end of the beam.

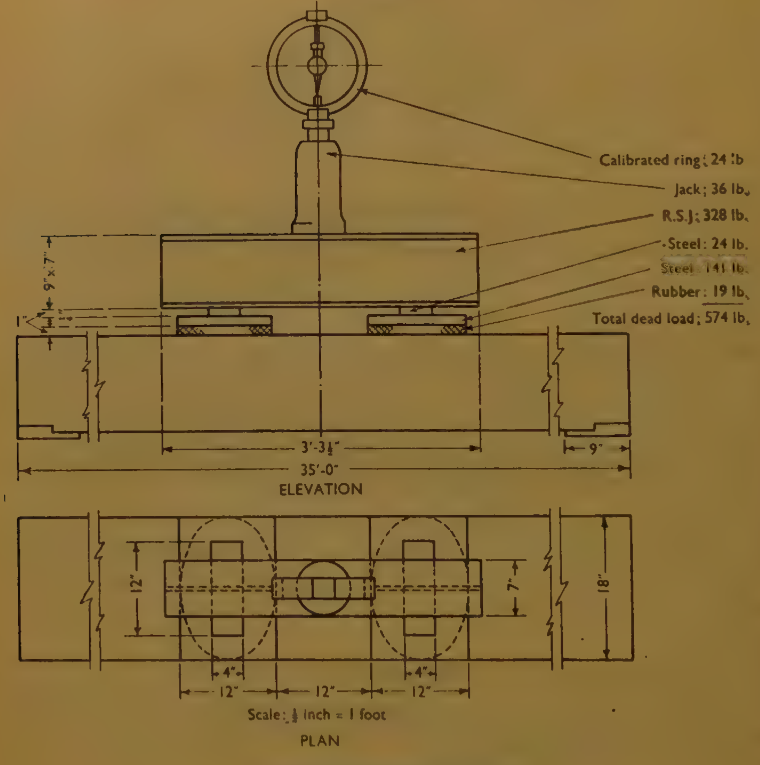
RESULTS

(1) *Pre-stressing*

Although some of these readings were not as consistent as had been wished, they produced some interesting results :—

As the pre-stressing force increased, the strains at the centre of the beam increased at almost the same rate as those at the bottom. This indicates that there was considerable restraint between the concrete bed and the beam itself. Towards the end of the pre-stressing the middle of the beam lifted and the bottom-fibre strains increased rapidly while the centre fibres remained almost constant. But the top-fibre strains (see *Fig. 25*, line 1) showed a negative value of 10×10^{-5} which is considerably greater than that calculated for the initial condition of pre-stressing.

Figs 21



ARRANGEMENTS FOR LOADING

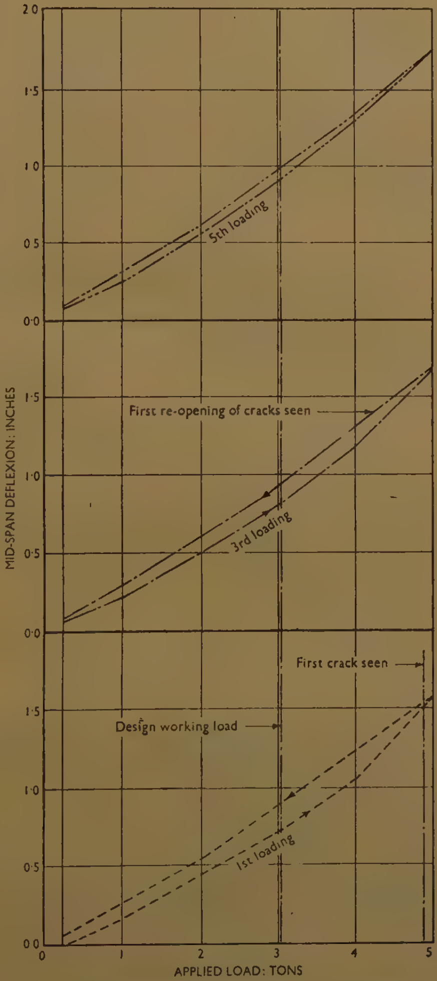
If, for this calculation only, a value of E of 5×10^6 lb. per square inch is assumed, then this strain corresponds to a tensile stress of — 500 lb. per square inch.

Immediately after stressing, the bottom-fibre strain was below that calculated (being only 25×10^{-5} instead of about 33×10^{-5}) and the top fibre was still as described above. One end of the beam was then lifted by a crane and replaced on its bed. The top fibre strain immediately dropped from -10×10^{-5} to -4×10^{-5}, which corresponds closely to the calculated value of — 215 lb. per square inch, but the bottom fibre increased from 25×10^{-5} to only 26×10^{-5} (*Fig. 25*, line 2). The extension of the cables, pressure-gauge readings, and anchorage slip were all satisfactory and no immediate explanation was obvious.

(2) *Loading Test*

Three weeks after stressing, the beam was placed in the test rig and strain readings taken again. The results of these (after allowance has been made for the weight of the testing equipment, jacks, etc.) is shown by line 3 in *Fig. 25*. This shows a decrease in the tensile strain in the top fibre which agrees very closely with calculations (approximately 50 lb. per square inch compression instead of a calculated zero), but the bottom fibre shows an *increase* in strain to 31 × 10^{-6} which, assuming in advance the value of E of 5·50 × 10^6 lb. per square inch obtained later, corresponds to a stress of 1,740 lb. per square inch. (The calculated value after all relaxations is 1,750.) It can only be assumed that either the strains had not distributed themselves in the concrete in the short time (about 30 seconds) between lifting the beam from its bed and replacing it, or that friction in the cables caused overstressing at the jacking end of the beam (where,

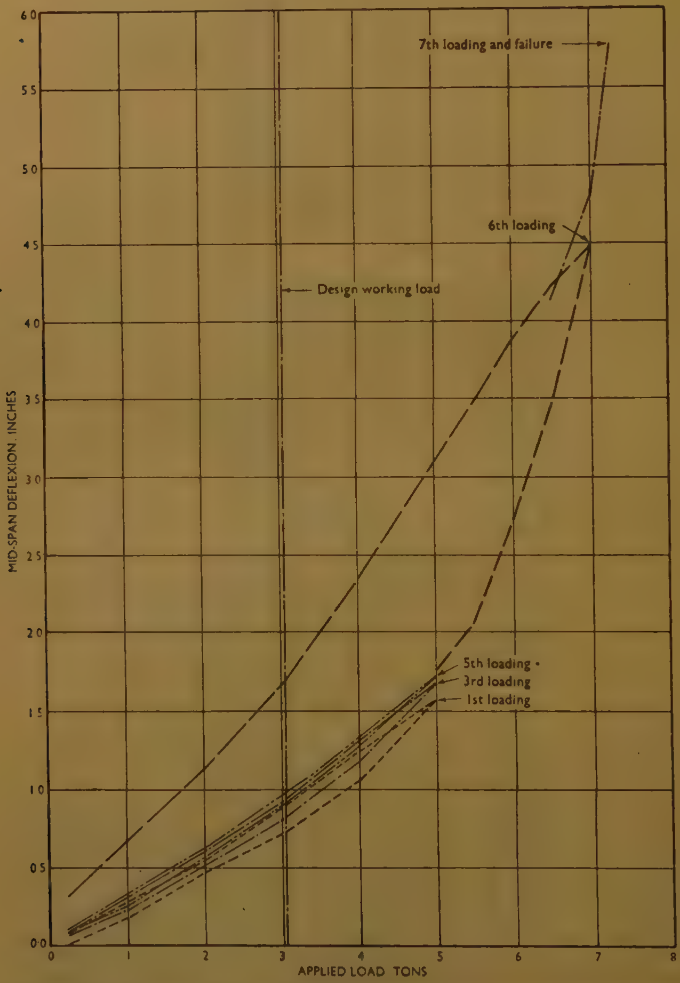
Fig. 23



LOAD/DEFLEXION CURVES SHOWING THE EFFECT OF REPEATED LOADS ON DEFLEXION AND ON CRACKING unfortunately, no readings were taken) and consequent understressing at mid-span. In the period between the stressing and the grouting of the cables these stresses re-distributed themselves to values in close agreement with the calculations.

It would have been of great interest to have taken frequent readings of the strains after the beam had been stressed, but this was not possible.

Fig. 24

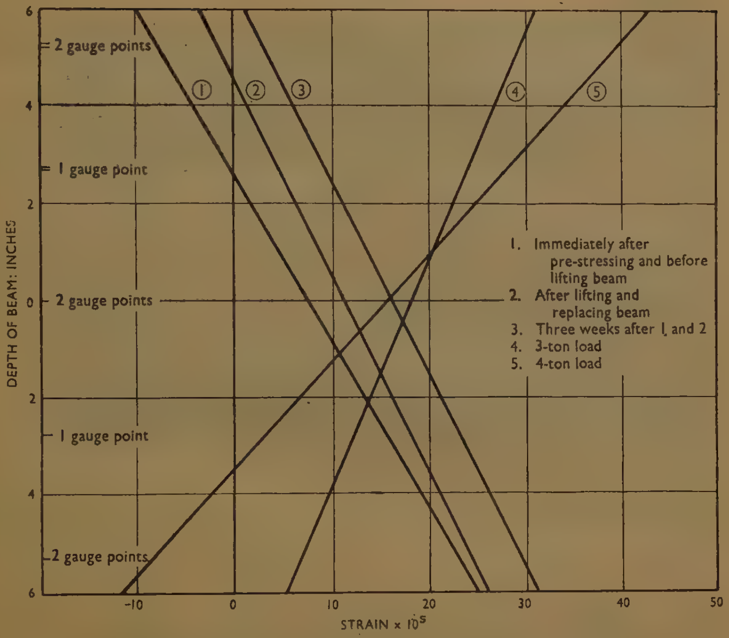


Load/Deflexion Curves Showing the Comparison between the Deflexions at Working Load and Failure

The load/deflexion curves (*Figs 23* and *24*) were plotted with the zero as unloaded condition of the beam in the test rig.

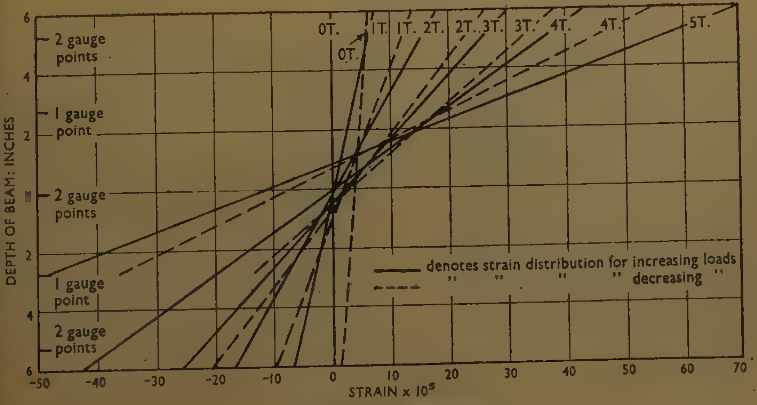
The load/strain curves were plotted in two ways; the first (*Fig. 26*) showing purely

Fig. 25



COMBINED PRE-STRESSING AND FLEXURAL STRAINS

Fig. 26



FLEXURAL STRAINS

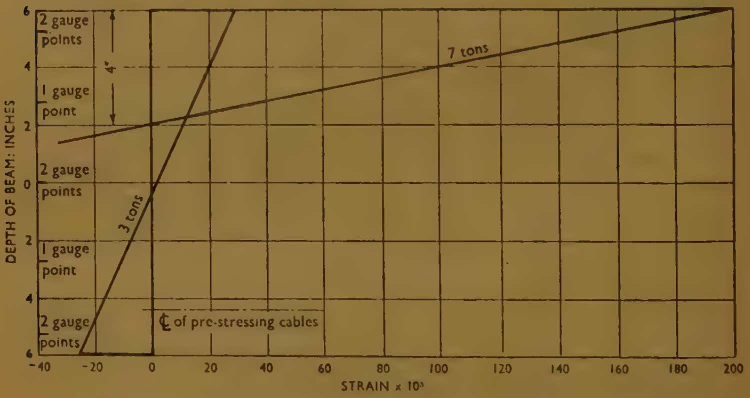
flexural strains, whilst the second (*Fig. 25*) combined these with the strains produced by pre-stressing.

The load/deflexion curves show very close approximation to a straight line until cracking point, when the curve rapidly steepens. This rate of increased deflexion agrees with the rate of rise of the neutral axis as shown by the strain diagrams in *Fig. 26*.

As the load was removed each value produced a slightly higher deflexion than the corresponding one on the increasing load. This was caused by creep taking place in the top concrete which, before the beam was loaded, had been virtually unstressed. The progressive closing of the gap between the "up" and "down" lines of successive loadings illustrates the decreasing rate of creep.

The beam was designed for a service load corresponding to a 3·06 tons point load and a load factor against cracking of 1·5. The first cracks appeared at 4·95 tons, which corresponds to a load factor of 1·65, and were clearly revealed by the whitewash that had been applied to the beam. The cracks were very evenly distributed at 6-inch to 8-inch intervals over a distance of 5 feet, indicating that good bond had been achieved by the grouting (this was confirmed after the beam had failed).

Fig. 27



Flexural Strains comparing 3-ton and 7-ton Loads

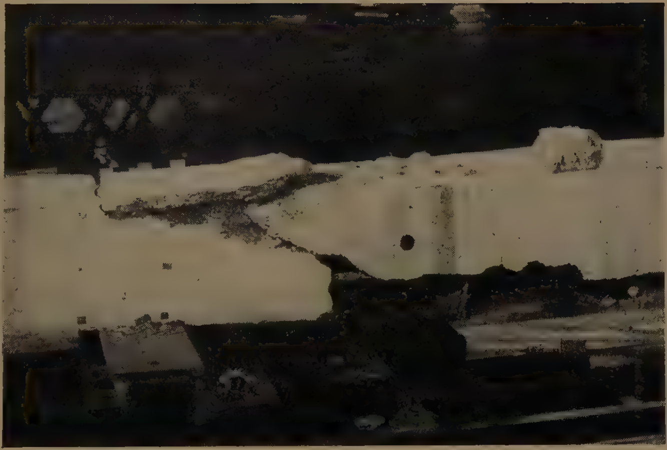
At the second loading the cracks reappeared at 4 tons, which produced a tensile strain in the bottom fibre of 10×10^{-6}. Even if it is assumed that the cracks opened invisibly at a slightly lower loading, this indicates that with a bonded cable the cracks do not re-open appreciably (if at all) immediately the bottom fibre stress becomes negative. With a bonded cable, the size of the crack depends on the strain of the steel at this point. If the bond is good then the length of wire free to strain is small, so that the cracks are fine and well distributed.

With increased loading, fresh cracks appeared symmetrically about the mid-span point, occasionally between cracks that had already opened but mostly beyond the initially cracked section. The first cracks gradually increased in depth; this also occurred symmetrically, the depth of crack corresponding proportionally to the vertical ordinates of the bending-moment diagram.

The beam was loaded to 7 tons and at this loading the cracks were well above the centre of the beam, corresponding to the position of the neutral axis shown by the strain diagram in *Fig. 27*.

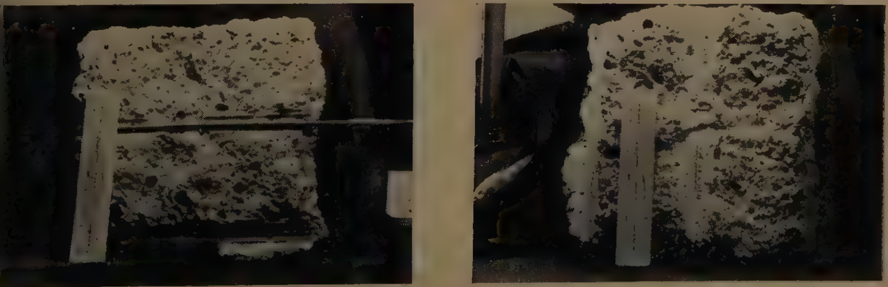
After progressive removal of this load, the residual deflexion was found to be only 0·31 inch or 14 per cent of the maximum under that load. This 7-ton load was found later to be 97·5 per cent of the ultimate, so the recovery to 14 per cent was very satisfactory. The recovery was measured immediately after the removal of the load, and would undoubtedly have been greater had the beam been left unloaded longer.

Fig. 29



Test Beam after Failure. The Tension Cracks to the Left of the Point of Failure have been closed by the Pre-stressing Force remaining in the Bonded Cable

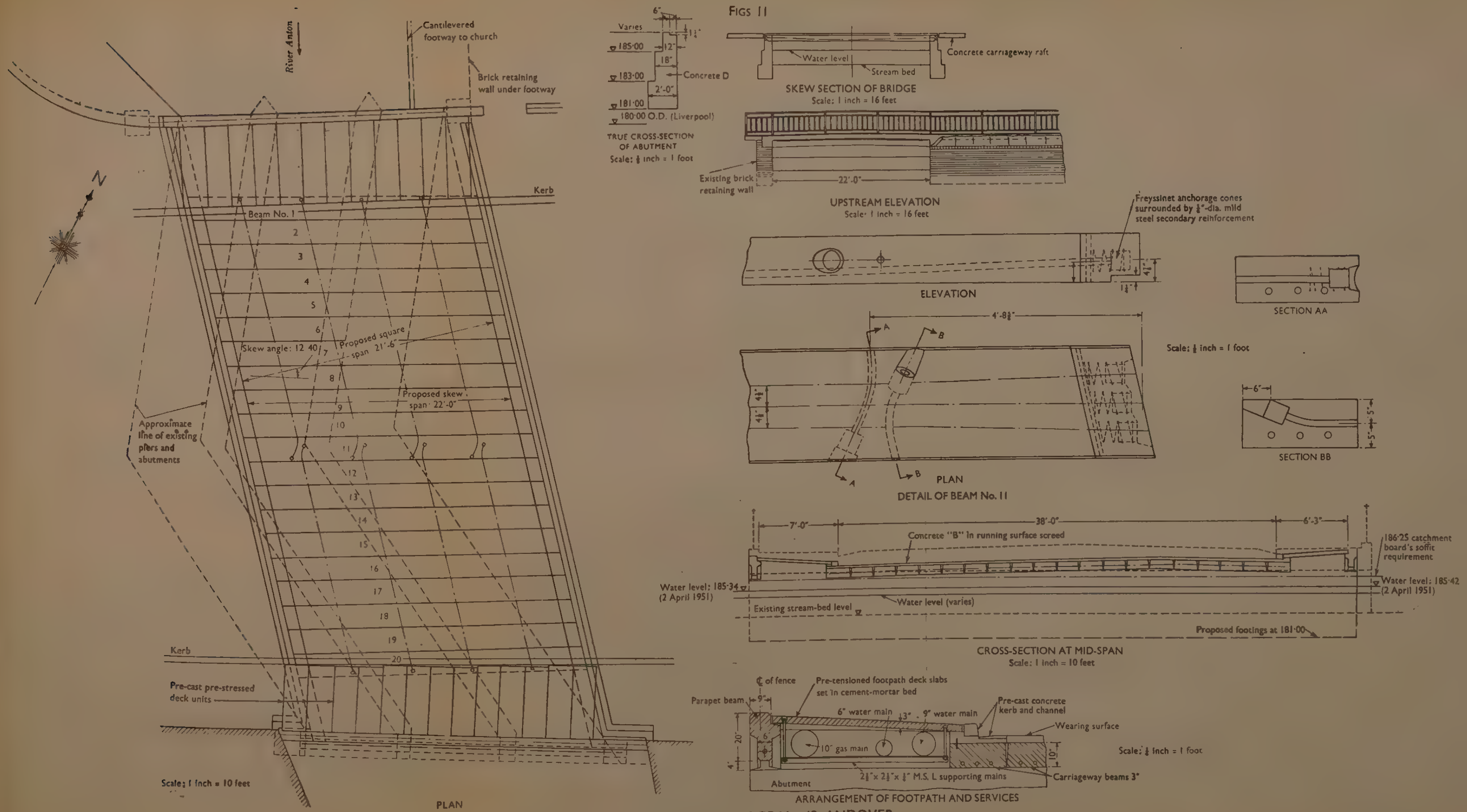
Figs 30



Section of the Bottom of the Beam, showing how the Grout had completely filled round the Wires

RECENT DEVELOPMENTS IN HIGHWAY BRIDGE DESIGN IN HAMPSHIRE

FIGS 11


River Anton
Cantilevered footway to church
Brick retaining wall under footway
N
Kerb
Beam No. 1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
Skew angle: 12 40
Proposed square span 21'-6"
Proposed skew span 22'-0"
Approximate line of existing piers and abutments
Kerb
Pre-cast pre-stressed deck units
Scale: 1 inch = 10 feet
PLAN
6"
Varies
1¼"
185·00
12"
18"
183·00
Concrete D
2'-0"
181·00
180·00 O.D. (Liverpool)
TRUE CROSS-SECTION OF ABUTMENT
Scale: ⅛ inch = 1 foot
Water level
Stream bed
Concrete carriageway raft
SKEW SECTION OF BRIDGE
Scale: 1 inch = 16 feet
Existing brick retaining wall
22'-0"
UPSTREAM ELEVATION
Scale: 1 inch = 16 feet
Freyssinet anchorage cones surrounded by ½"-dia. mild steel secondary reinforcement
ELEVATION
SECTION AA
4'-8½"
A
B
4½"
PLAN
DETAIL OF BEAM No. 11
Scale: ½ inch = 1 foot
6"
5"
SECTION BB
7'-0"
38'-0"
6'-3"
Concrete "B" in running surface screed
186·25 catchment board's soffit requirement
Water level: 185·34 (2 April 1951)
Water level: 185·42 (2 April 1951)
Water level (varies)
Existing stream-bed level
Proposed footings at 181·00
CROSS-SECTION AT MID-SPAN
Scale: 1 inch = 10 feet
℄ of fence
Pre-tensioned footpath deck slabs set in cement-mortar bed
Parapet beam
9"
6" water main
3"
9" water main
Pre-cast concrete kerb and channel
Wearing surface
20"
6"
4"
10" gas main
10"
Scale: ½ inch = 1 foot
2½" x 2½" x ½" M.S. L supporting mains
Carriageway beams 3"
Abutment
ARRANGEMENT OF FOOTPATH AND SERVICES

RECONSTRUCTION OF TOWN BRIDGE No. 48, ANDOVER

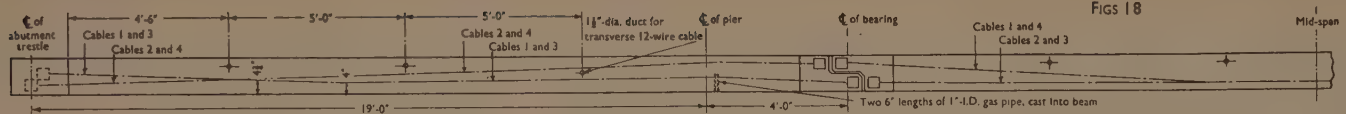
The Institution of Civil Engineers   Proceedings, Part II, June 1952

E. W. H. GIFFORD

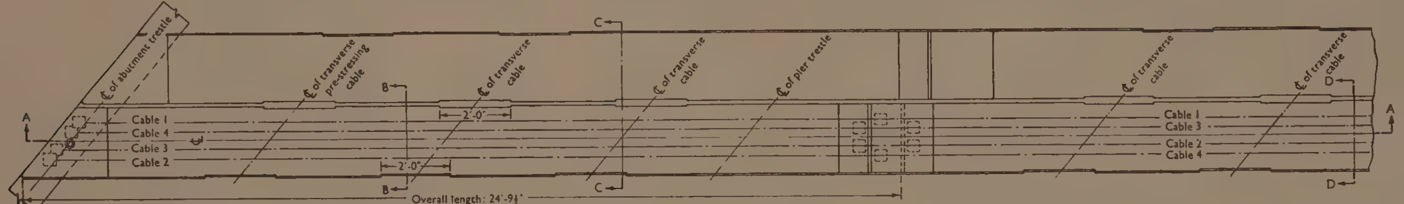
William Clowes & Sons, Limited: London

RECENT DEVELOPMENTS IN HIGHWAY BRIDGE DESIGN IN HAMPSHIRE

Figs 18

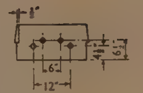


LONGITUDINAL SECTION AA



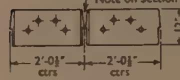
PART PLAN SHOWING TWO ROWS OF BEAMS
Scale: ½ inch = 1 foot

24"x 6" pockets are formed in sides of beams at each transverse pre-stressing cable. These pockets are filled with concrete as specified before pre-stressing. After pre-stressing, the remaining lengths of longitudinal joints will be filled with fine concrete

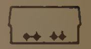


SECTION BB

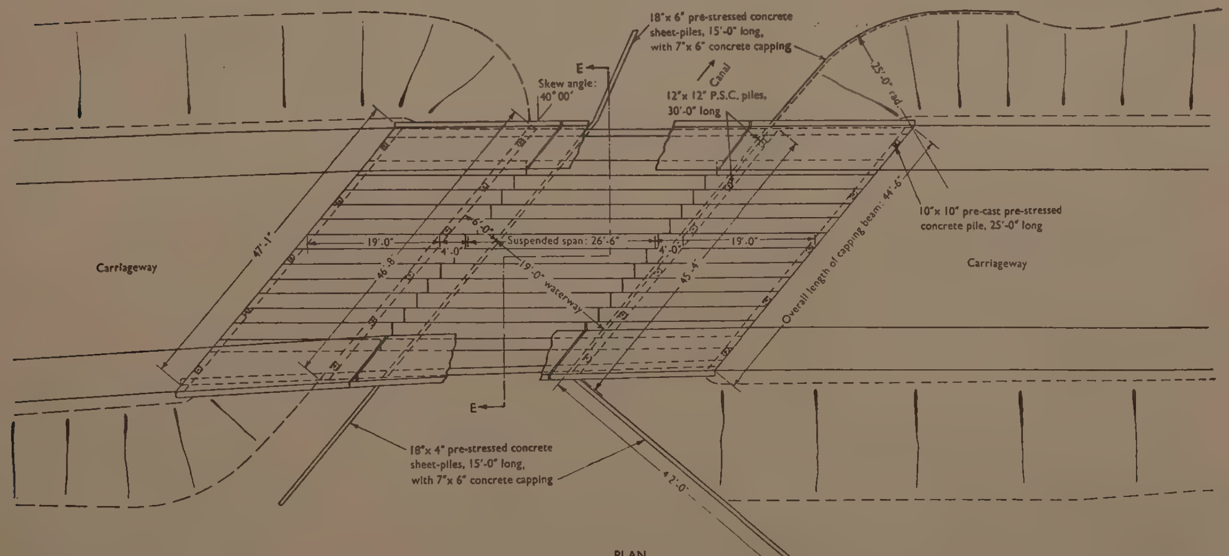
24"x 12" pocket formed at the transverse cable adjacent to the piers. Filling is described in Note on Section BB



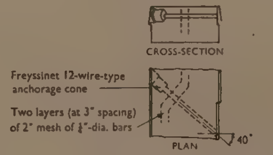
SECTION CC



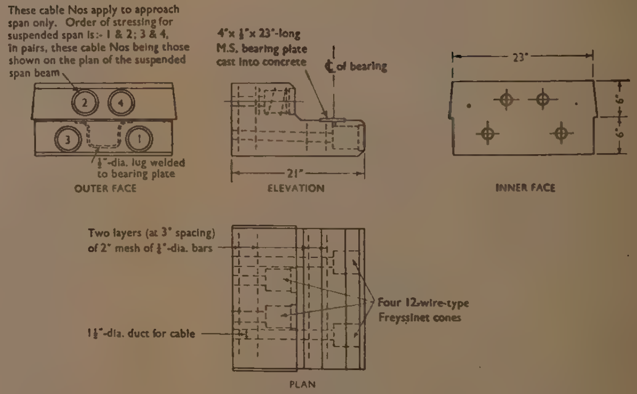
SECTION DD


18"x 6" pre-stressed concrete sheet-piles, 15'-0" long, with 7"x 6" concrete capping
Skew angle: 40° 00'
E
E
Canal
12"x 12" P.S.C. piles, 30'-0" long
25'-0" rad.
10"x 10" pre-cast pre-stressed concrete pile, 25'-0" long
Carriageway
Carriageway
47'-1"
46'-8"
45'-4"
19'-0"
4'-0"
6'-0"
Suspended span: 26'-6"
4'-0"
19'-0"
19'-0" waterway
Overall length of capping beam: 44'-6"
18"x 4" pre-stressed concrete sheet-piles, 15'-0" long, with 7"x 6" concrete capping
42'-0"

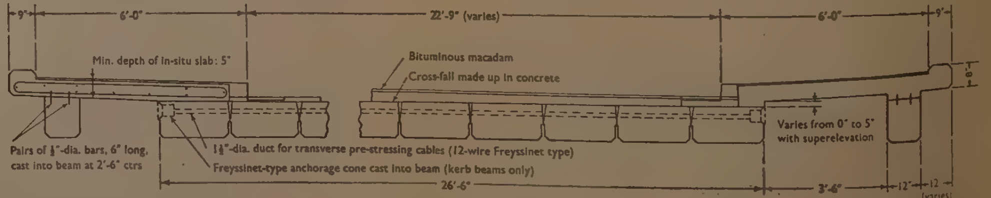
PLAN
Scale: 1 inch = 20 feet


CROSS-SECTION
Freyssinet 12-wire-type anchorage cone
Two layers (at 3" spacing) of 2" mesh of ⅛"-dia. bars
PLAN
40°

DETAILS OF TYPICAL ARRANGEMENT OF TRANSVERSE ANCHORAGE
Scale: ½ inch = 1 foot


These cable Nos apply to approach span only. Order of stressing for suspended span is:- 1 & 2; 3 & 4, in pairs, these cable Nos being those shown on the plan of the suspended span beam
2
4
3
1
⅛"-dia. lug welded to bearing plate
OUTER FACE
4"x ⅛"x 23"-long M.S. bearing plate cast into concrete
℄ of bearing
21"
ELEVATION
23"
6"
6"
INNER FACE
Two layers (at 3" spacing) of 2" mesh of ⅛"-dia. bars
Four 12-wire-type Freyssinet cones
1½"-dia. duct for cable
PLAN

DETAILS OF SUSPENDED SPAN BEARING ANCHORAGE UNITS
Scale: ½ inch = 1 foot


9"
6'-0"
22'-9" (varies)
6'-0"
9"
8"
Min. depth of in-situ slab: 5"
Bituminous macadam
Cross-fall made up in concrete
Varies from 0" to 5" with superelevation
Pairs of ⅛"-dia. bars, 6" long, cast into beam at 2'-6" ctrs
1½"-dia. duct for transverse pre-stressing cables (12-wire Freyssinet type)
Freyssinet-type anchorage cone cast into beam (kerb beams only)
26'-6"
3'-6"
12"
12 (varies)

CROSS-SECTION EE OF CENTRAL SPAN
Scale: ½ inch = 1 foot

RECONSTRUCTION OF PONDTAIL BRIDGE

E. W. H. GIFFORD

The Institution of Civil Engineers Proceedings, Part II, June 1952

William Clowes & Sons, Limited: London

All the cracks completely closed so that they were quite invisible, except for one or two where very slight spalling of the whitewash revealed their presence.

The beam was again loaded to 7 tons and a slightly higher maximum deflexion was obtained, which increased under constant load for a short period before steadying on the value shown in *Fig. 24*. This 7-ton load represents a load factor of 2·33 and even after this repeated load, the beam remained perfectly serviceable.

The load was then increased to 7·23 tons, when the beam failed with a loud report by crushing of the top concrete (*Fig. 29*). The deflexion was approximately 5·7 inches and the load factor 2·40.

Failure occurred at the point where the first cracks were most pronounced (*Fig. 28*, facing p. 475), 18 inches from mid-span and beneath one of the loading points. After failure the beam dropped on to the safety packing, one half remaining intact. Immediately all the cracks in the bottom of the beam closed and fresh cracks appeared in the top. This shows that the cable in the undamaged portion of the beam was still bonded (see *Figs 30*) and the top-fibre tension produced by the pre-stress was no longer being balanced by the dead-load bending moment of the beam. The cracks produced were deep and regular and it is interesting to note that the bond was still so good after repeated overloading and the considerable shock of failure. No special precautions were taken for the grouting, a simple hand-pump was used to inject the neat cement grout, which had no admixtures.

DETERMINATION OF VALUES FOR E

These were calculated in two ways, firstly from the deflexions and secondly from the measured strains, with the following average results:—

From deflexions: $5{\cdot}6 \times 10^6$ lb. per square inch.
From strains: $5{\cdot}4 \times 10^6$ lb. per square inch.

The mean value of 5·5 is reasonable for 5-weeks-old high-quality concrete cast in a medium-sized section.

CONCLUSION

Because of its brevity this simple test cannot be considered as absolute proof of the design assumptions and load factors chosen, but it strongly supports them.

Discussion

The Author introduced the Paper with the aid of a series of lantern slides and a film. He remarked that, since the MS. of the Paper had been prepared, live-load tests had been carried out on Bickton footbridge. The results were as follows:

1. The static live-load test showed a very small deflexion, equivalent to about 1 inch under full live load (a deflexion/span ratio of 1/840).
2. Despite the stiffness shown by test 1, it was possible for six people jumping together to work up a sympathetic vibration with an amplitude of 1¼ inch and a frequency of 2·7 cycles per second. Although there could be no danger to the structure, that state of affairs was not desirable and the vibrations had been damped by hanging weights on springs to oscillate out of phase with the bridge itself.

Messrs Crockett and Hammond[1] had suggested that structures could be placed on vertical springs or dashpots. In the case of a bridge, they could be built into the abutment bearings and, apart from the question of maintenance, would be a simple solution. The Author would be pleased to hear from other engineers who had had similar experience with lightly loaded structures.

Mr A. J. Harris observed that the Author, with great frankness, had referred to the vibrations which had been noted on the bridge of 70 feet span. That was extremely interesting to anyone concerned with pre-stressed concrete, and it was a triumph of a kind to be able to build anything in concrete which was light enough to vibrate. Normally almost anything could be done to such a concrete structure before the slightest quiver was felt. It seemed as though with pre-stressed concrete the stage was almost being reached when care had to be exercised not to make a structure too light. The depth/span ratio on the Author's bridge was about 1 in 35, which was rather more slender than any simply-supported span had yet been, but the interesting point was that while the static deflexion was not large there did arise vibrations which were appreciable. Taking the Author's figures of the frequency and amplitude of vibration, and using Pain and Upstone's formula,[2] the maximum acceleration could be computed as approximately 6 feet per second per second. Mr Penn, in a Paper written jointly with another engineer some time ago which dealt with factors limiting the depth of highway and other bridges, had indicated that accelerations in excess of 4 feet per second per second were apt to be noticeable.

Some information was available on pre-stressed concrete bridges having a span/depth ratio of the same order. Mr Harris had some data on a highway bridge built in France, about 1940, which had a span of 66 feet and a depth of a little more than 2 feet. Dynamic testing of bridges was more common in France than in Britain and in the instance in question lorries had been run over the bridge in both directions and over planks 3 inches thick; the vibration had been registered and frequency of about 4 cycles per second had been indicated. That was faster than the Author's bridge. The amplitude of oscillation under an 18-ton lorry falling off a 3-inch plank was 3 millimetres, which was not particularly large.

There would appear to be a question of principle involved. How important were those vibrations and deflexions? The case often brought forward was that of a column of troops marching. The period of the Author's bridge was fairly near that of marching troops, which was about 140 to the minute, while the Author's figure was about 160. Was that a cause for concern ? Were bodies of men three abreast expected to cross a bridge of the nature in question ?

---

[1] J. H. A. Crockett and R. E. Rolt Hammond, "Reduction of Ground Vibrations into Structures." Structural Paper No. 18, Instn Civ. Engrs, 1947.

[2] J. F. Pain and T. J. Upstone, "Some Considerations affecting the Minimum Depth of Small Highway-Bridge Girders." Road Paper No. 10, Instn Civ. Engrs, 1943.

In his quiet way the Author, under the aegis of Brigadier Hughes, had done something rather remarkable : he had built very slender footbridges, of quite appreciable span, out of pre-cast elements all of which were exactly the same. He had also done it economically. Mr Harris did not suggest that that made history, but it was a useful achievement. The hollow block elements, which the Author had produced economically for four bridges in different sites, seemed to be an appropriate element of construction for mass production.

Mr Harris had been a most interested spectator of the construction of those bridges. They were the Author's design, but the Author had been good enough to ask him occasionally what he thought about them. The Author had occasionally designed to figures below the minimum thicknesses which Mr Harris's company would care to recommend to somebody else, but the construction of the bridges had been carried out, under the Author's supervision, remarkably well, and a number of problems which that type of construction raised had been solved by him in a particularly ingenious manner. In particular Mr Harris liked the "toothpaste machine," which was a very successful little piece of plant.

Mr F. J. Samuely said that he had visited the sites of the bridges described in the Paper 3 or 4 months previously and had been so impressed that he had immediately altered the design of a bridge on which he had then been working.

He was not quite sure whether it was worth while using high-tensile steel in pre-stressing for the transverse reinforcement, which was usually rather short. The object of that transverse reinforcement was, of course, to ensure that if the load were concentrated on one beam then all the beams, or at least several of them, would carry it together. Since the concrete stresses across the bridge were rather small, he suspected that it was sufficient to use an ordinary long bolt of mild steel and to tighten the nut. The horizontal creep would be very small, because of the small stresses.

Mr Samuely emphasized the well-known fact that pre-stressing need not be done in one straight line. As the Author's bridges showed, it was quite possible to pre-stress in a curve, and if the difficulty of friction could be overcome it would be possible eventually to pre-stress in a curve of much smaller radius.

Mr J. E. Jones said that he had noticed vibration, of the kind that the Author had mentioned, on a highway bridge constructed of pre-stressed beam units—not transversely pre-stressed, but of I-section, of 51 feet span and 2 feet 3 inches deep. There was noticeable vibration under certain conditions as traffic crossed the bridge. No measurements had been taken, so that he could not say what sort of period the vibrations had nor what their amplitude might be. The idea of suspending a spring-borne mass on the bridge to damp out vibrations had been mentioned some years previously by Sir Charles Inglis,[1] who had experimented with some form of truss railway structure of 150- or 200-foot span. A mass of about 10 tons, spring borne, had been suspended under the bridge and had very effectively damped out the vibrations (from locomotive hammer-blow in particular) that were introduced.

What did the Author mean by a "crippling" load in the third paragraph on p. 464, where he stated that a reasonable factor of safety against crippling would be 2½. Mr Jones did not quarrel with the 2½, but would like to know just how a crippling load was defined; he found it a little difficult to appreciate what that would be for a pre-stressed structure. He agreed with the Author that a factor of safety of about 1½ against cracking would be a reasonable criterion to adopt in design.

Some tests had been carried out on beams of the type to which Mr Jones had referred earlier; they were of I-section and had been tested at the age of 10 years. He hoped that it might be possible, when the results had been properly reviewed to publish something about those tests, which were interesting on account of the age of the specimens. The point which he wanted to make now, in connexion with the suggested factor of safety of 1½ against cracking, was that in the case of two beams which had been tested the factor of safety in one instance had proved to be exactly 1·4 and in the other a little more. That seemed to bear out the reasonableness of the Author's suggestion of 1½.

Mr Jones had been a little surprised to discover, from p. 467, that in the case of the beam which had been tested the cables had been initially tensioned to 69,000 lb., which worked out at about 82 tons per square inch, and that after losses had taken place the residual stress in them was 72 tons per square inch. A figure of 82 tons per square inch seemed to be higher than the value of the proof stress for the particular 0·2-inch wire used, and he could not help feeling that the 72 tons per square inch which was said to be the final load was too near the proof stress. He thought that a residual stress of 60–65 tons per square inch would have been more prudent. It might be, however, that already he was getting a little old fashioned so far as that matter was concerned.

He did not think anyone knew a great deal about how to use pre-stressed concrete to the best advantage and what type of construction was best suited for a particular job. He wondered whether the solid slab unit which was shown for some of the beams was, after all, the best sort of unit. Furthermore, was it really worth while undertaking transverse pre-stressing to the extent to which it had been done in the bridges described in the Paper? That that was not a criticism of what had been done in Hampshire, but a request for the Author's view, because at the moment too little was known about transverse pre-stressing.

---

[1] "Mechanical Vibrations: Their Cause and Prevention." James Forrest Lect., J. Instn Civ. Engrs, vol. 22, p. 312 (Oct. 1944). See p. 348.

Mr H. W. Hayton raised the question of a possible alteration in elevation of the 70-foot-span footbridge. If the Author had wished to make the rise say 2 feet higher than in the drawing, could he have done it by so arranging his centering from the start that the bridge centre was 2 feet higher? Would the same pre-cast sections be suitable for the bridge in every other respect?

Mr D. E. Hennessey questioned the Author's statement that an essential feature of bridge building was speedy construction. Mr Hennessey said that he spoke in general terms because he did not know local conditions in Hampshire, but he thought that there was a case for taking one's time over building a bridge. Most new bridges to-day would either be reconstructions on an existing traffic line, and usually in such a case they would be small, or they would be large bridges built on trunk roads with a completely new alignment. In the first case diversions over small streams could be accomplished by methods developed during the war, using, say, Armco tubing and bull-dozing in.

He mentioned that because it led to another point which the Author had raised—the alternative methods of pre-tensioning and pre-casting bridge units rather than post-tensioning them. In large bridges in particular a much smaller section was usually obtained by post-tensioning, and the dead-to-live-load ratio was important. In that respect had the Author considered post-tensioning on his larger bridges and would it have made any difference to his sections?

With regard to transverse tensioning, had the Author any figures about the stresses induced in a transverse direction, as compared with the longitudinal stresses in the concrete? Mr Hennessey had found that transverse tensioning would, as Mr Samuely had pointed out, make it possible to regard the deck as a slab, and he thought that for the Ministry of Transport loading, and particularly the abnormal loading, the advantage of being able to distribute the load over the whole deck, or a large portion of the deck, was quite considerable, when it was measured in terms of the depth of member used.

Turning to the general appearance of the bridges, as shown in the photographs, the large hump in the road bridge was not, he thought, commendable as a general rule; the emphasis should be on making the bridge a part of the road design, to keep it as a unit. On the other hand, the hump, or some indication of its purpose, would be better in some of the footbridges, which in the photographs were shown as being straight across. He was not too happy about the thickness of the members, in the footbridges in particular. He presumed that in the New Forest the site was almost enclosed; it was a localized scene, and as much lightness as possible should be obtained in the design. He would like to see the bridge thinned down towards the centre of the span, if that were technically possible, and he thought that pre-stressed concrete was the best method of achieving that.

It seemed probable that in larger bridges engineers would be asked to produce much more slender designs and he thought that pre-stressed concrete was the new medium for doing so.

Mr Robert Saunders asked if any trouble was likely to occur on the 70-foot-span footbridge from numbers of hikers leaning on the rail? It seemed to be anchored very low down and not to have very great resistance to torsion. On what might be called the square tubular footbridges, had the Author considered, instead of putting the steel between the two units, putting it through the middle? The use had been suggested of standard concrete pipes without flange ends, the steel running through the middle and anchored by means of diaphragms. One of the difficulties of that method had been to avoid corrosion of the steel. Could the Author make any suggestions about that?

In stressing the square-tube units, there was a tendency for the tubes to buckle sideways. Presumably that was resisted by the diaphragm units. Had that been designed for?

What was the criterion for transverse pre-stressing and how did one know how much to tighten up in order to get distribution?

Had there been trouble with friction on the pins? In the design which Mr Saunders had suggested, using tubes, it was suspected that with steel bars, if not with cables, there might be trouble with friction on the locating pins.

In the picture of one of the road bridges it was possible to see the top of the members; there was no wearing course on the road. Had any extra depth been allowed there, not counted in the calculation, to allow for wear?

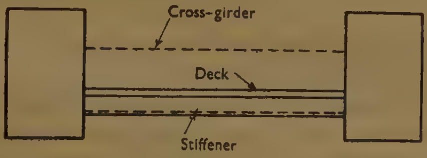
Mr G. O. Kee asked whether any other, alternative methods of construction had been considered. Pre-cast work was very attractive, but usually it proved somewhat uneconomic, presumably largely because of factory overhead costs. Had any comparative costs been obtained for composite construction similar to that used by, for example, the Eastern Region of British Railways—inverted T-beams, or pre-stressed beams with in-situ decks? He had usually found when making cost comparisons for road bridges of medium spans, say 50 feet and more, that beams at 6-foot centres and in-situ decking worked out as the cheapest, but they were of rather deep construction and usually a little unsightly.

Mr J. Maclagan Smith referring to the designs given in the Paper for future works, asked if the footways had been designed for an accidental live load. In the case of the Test Bridge, Stockbridge, the footway appeared to be of extremely light construction. Was the footway slab between the outer supporting beams designed for a 4-ton wheel load?

Referring to the vibration mentioned by the Author, Mr Smith commented on the film of one of the road bridges which showed traffic running on the beams without any road surfacing. Did the Author advocate that? Would it not have been better to resort to the conservatism of the past and have a damp-proof course with, perhaps, a 2-inch concrete cover and tarmacadam or asphalt surfacing on top? Did the Author consider that a damp-proof course over a pre-stressed concrete deck was unnecessary?

Mr R. C. Harvey referred to the problem of vibration on the 70-foot span footbridge. The structure in cross-section seemed to be simply a pair of beams of roughly rectangular section, with the deck slung between them and resting on shallow stiffeners, as shown in *Fig. 31.* Could the Author say whether the vibration caused by six men jumping up and down in unison, occurred on both girders at the same time, and whether the amplitude, positive and negative, was also the same in both girders at the same time. If it was not, it seemed to indicate that the design was somewhat defective from the cross-girder point of view. Mr Harvey's own principle when designing bridges had always been to endeavour to avoid vibration at any cost; he thought that cross-girders rigidly connected to the main girders, which would mean increasing the depth and lifting up the decking, would contribute to a reduction of the vibration.

Fig. 31



The design in question was a very bold one, on which the Author was to be complimented, but the reduction in weight might have been overdone. In making observations on the vibration, had the Author carried out any experiments by adding weight in the form of, say, sandbags distributed along the span at intervals? The Author had not much spare live load to play with, but an increase in weight on the structure might help. One way of putting on a little weight was to increase the wearing carpet; perhaps the Author would consider that.

Mr J. M. Campbell, referring to the pre-cast capping beam mentioned on p. 469 and shown in *Fig. 10,* observed that the cable seemed to have a very sharp curvature. Had the Author taken friction into account and had he in this case stressed from both ends? The view was apparently held that friction played a considerable part in the loss of stress.

In *Figs 19* the sections of the beam showed a very thin lower flange occupied by seven Freyssinet cables. The actual consolidating of concrete in that flange must have been rather difficult. The Author could, of course, turn the beam upside down, but presumably he wanted the good face to the bottom flange. Moreover, the diaphragms appeared to be rather finnicky little things from the point of view of the formwork.

The Chairman commented on a few practical points. The Paper described, he said, what was undoubtedly a very economical method of constructing bridges; he was quite satisfied about that now. The footbridge units were now a stock item. The County Surveyor's Department of Hampshire had, like many other Highway Authorities, taken over a very long mileage of country footpaths. There were three very famous rivers in Hampshire, with branches, and the provision of long-span footbridges across those rivers presented a problem, because the Rivers Board did not like any intervening obstacles in the form of mid-span supports. To be able to stock the footbridge units and use them as required had been of very great advantage.

The Rhinefield Bridge, partly pre-stressed concrete, partly post-stressed concrete, had cost £975, compared with an actual tender for the conventional design of £1,750. That was because it had been possible to save all under-water work by using a longer span and avoiding abutments. With regard to appearance, the public at large were really very pleased with the bridge.

The Author, in reply, referred to the question of troops crossing the 70-foot-span bridge. County Surveyors were worried less about troops than about a few hooligans. The old suspension bridge which the present bridge had replaced had been known locally as "the swing bridge," and with good reason, and Brigadier Hughes feared that the present bridge might be called "the bouncing bridge."

The important fact about transverse pre-stressing was that the comparatively narrow beam of about 18 inches in width could, if the whole structure were united, be designed for a uniformly distributed load, and a very heavy concentrated wheel load need not be taken into account except in the design of the transverse pre-stress. His principle in that case was to put in sufficient transverse pre-stress, plus a little more, to take account of the shear between two adjacent beams. He thought that an ordinary tension bar would not be sufficient because there would then be a direct shear on the circular bar, the effect of the load on the whole of the surface of the beam would not be uniformly spread. He usually took a very conservative figure for the coefficient of friction—about ½. He was told that that was very small, and that the coefficient between two concrete faces under those conditions was greater than unity. He allowed about ½ and put in enough pre-stressing force to overcome that.

On the question of pre-stressing in curves, those bridges were in the main designed for straight beams, with a check on the curved-beam theory from which he found almost no deviation at all, and the only thing to take into consideration was the stability of the beam in handling, if indeed the beams were to be handled and not cast in situ. He had tried to persuade the contractor to assemble the Bickton Bridge on the bank and then to launch it, but he would not do it.

Friction round the bend must also be considered. In the bridge to which he had just referred the friction was of the order of 5 per cent, which was very small. There advantage had been taken of the curve by running a cable from the top down to the bottom to that the cable ran very nearly straight along the beam. In a beam which was, say, 2 feet deep, with a 1-foot rise, the cables would run almost horizontally and there would be less friction.

The Author felt honoured that his work had been bracketed with that of Sir Charles Inglis, but Sir Charles had hung 10 tons of steel in the very best engineering fashion, whereas the Author had hung a couple of kerbstones. Eventually it had been found that one kerbstone was enough, with a weight of about 1½ cwt, whereas the applied force of six men had been considerably more than that. The addition of static dead weight would not solve the vibration problem. It would serve only to reduce the frequency and so, in the particular case of the footbridge, would make the structure more susceptible.

The Author believed that the cost of an in-situ deck, with mesh reinforcement to distribute the loads, would be greater in terms of overall economy. The additional strength required to carry the load and the consequently additional construction depth would be much more costly than transverse pre-stressing cables.

" Crippling load " could not, of course, be defined exactly ; his idea in introducing the question of the crippling load had been to get some comparison between the state of a reinforced-concrete beam at the stage of yield in the steel and similar conditions in pre-stressed concrete. Perhaps it would be clearer if he said that he had used the term " crippling load " in the sense that it was the worst condition of loading under which the beam was still just serviceable. That was vague, but definitions of load factors and factors of safety were always somewhat vague.

He agreed with Mr Jones that for bridges which were liable to carry heavy loads it would not be advisable to exceed 65 tons tension in the cables. The bridges in question, however, were for very lightly-trafficked unclassified roads, and it had been found when testing one of the beams that the steel did not fail, and that failure took place in the concrete. He agreed, however, that it was as well to be conservative on that question, particularly on road bridges of any importance.

On the question of whether solid slabs were the most economical to use, he thought that in the case in question they were. A fair amount of experience had been gained in Hampshire with the hollow form of construction for footbridges, but those footbridges might have been cheaper built in a solid unit if it had been possible to get the solid unit to the site. The sites in question, however, did not lend themselves to the transport of heavy units. On small spans, up to 35 feet, solid slabs would probably be cheaper, but if mass production of the hollow type of unit were developed, costs might be reduced. The hollow unit was not unduly expensive if used in the correct place.

It was certainly possible to increase the rise of the 70-foot-span bridge using the same units, provided that allowance were made for a slight increase (possibly 5–10 per cent) in the friction of the cables. The Author did not intend to enter into the friction-in-cable controversy which had raged rather fiercely in pre-stressed concrete circles recently. With the particular units in question, if the rise was increased it would probably be advisable to chamfer the ends of the 6-foot sections so that they fitted the joint. In fact, they had been finished off square, so that the joint tapered, but with more rise it would be more satisfactory from the point of view of the joint to cut off the ends of the beams slightly so that they approximated to a voussoir section. The beam would require bracing until both beams were united by the decking or transverse braces.

With regard to the effectiveness of the transverse braces in dealing with vibration, efforts had been made to vibrate two girders out of phase, but it could not be done. The transverse braces certainly did look very slender, but the calculations were correct, and in point of fact it was impossible to separate the two beams, which vibrated exactly in unison. It seemed, therefore, that the transverse units were very effective. They were stressed across by means of a bar, so that there was some permanent compression in the brace itself. On the question of the stability of the handrail, he thought that the answer was that those braces were sufficiently rigid, and it would be a simple calculation to work out the bending moment induced in them by a given number of hikers exerting a given number of pounds each. If any doubt existed it would be possible to fit a handrail in such a way that if people pushed really hard it would come off.

The question of speed of construction was relative to the particular job. The little New Forest bridges described were on single-line roads where there was no possibility of two-way traffic over the bridge during construction. A temporary bridge would probably have cost almost as much as the final bridge. There was not much to play with in £900 for temporary works. So far as two-way traffic was concerned, on busy roads, where the existing road was usually adequate for what it carried, if it were cut in half for a period of several months it would cost the Authority nothing, but would probably cost the public a great deal.

On the question of post-tensioning or pre-tensioning, the units had all been pre-cast, but they had all been post-tensioned with the exception of footbridge planks and apart from those he had not used pre-tensioning at all. On spans of any length it would appear that post-tensioning was more economical, by reason of the fact that it was possible to take account of the deadweight of the beam. On the question of transverse tensioning, it was not so much a matter of what was the stress in the concrete but of what was the shear between the beams, though on bridges of great width and small span it was necessary to take into account the transverse bending produced by the heavy load. However, that was something which could be evaluated by the usual methods with some degree of accuracy, and there was nothing particularly difficult about it.

With regard to general appearance, it had been suggested that the bridge should fit in with the road. That was so where the road was the predominant feature, but in the case of the Rhinefield Bridge the road barely existed, and the forest was the predominant feature, so that it had been felt that the bridge should fit in with the forest rather than the road. The footbridges were not straight across; the rise with the "rather flat footbridges" had been exactly the same as that of the "rather humpy road bridge"—12 inches in 35 feet. It was probable that the photographs in the Paper did not show that clearly, but the drawings did. From the point of view of slenderness, the 43-foot-span footbridge had a span/depth ratio of 45/1, which was going quite far enough without the introduction of fixed arches; that was not economical with small spans, and not very economical with pre-stressed concrete in any case. From the point of view of sheer economy, the simply-supported arrangement had an advantage for structures of the kind in question.

Putting the pre-stressing cable through the middle of the unit raised corrosion problems. If the steel were put down the middle of a hole, the hole had to be filled in solid or some other method had to be devised for protecting the cable. It might be possible to enclose the cable in plastic sheath, which was a reasonably cheap material, and grout down the centre of the sheath. In that way it would be possible to protect the cable without having to fill up the whole area. Grouting gave excellent protection. The disadvantage of that would be that the cable would not be bonded to the main concrete, which would reduce the ultimate factor of safety (see Appendix) for point loading. For light footbridges that might not be very important. Sideways buckling had been taken care of by the stiffeners. It was of course true that when a pair of units was stressed by a cable running between there was a tendency for the load to be higher in the adjoining walls of the blocks than in the outer walls. That was taken care of by the bar which joined the two sections. The units were not free to strain laterally, and there was a uniform distribution of stress along the unit.

The friction on the pin was very small—smaller than the friction in ordinary ducts—if the cable was not made to go round too sharp a curve.

Several speakers had been misled by the film and by the photographs in the Paper into believing that a wearing course was omitted. The pictures had shown the bridge only partly completed and a wearing course had indeed been used. The normal standard was 1½ inch minimum, and 2 inches of asphalt on trunk roads and other roads carrying heavy traffic. He thought that the transverse pre-stress was one of the best guarantees of a waterproof deck. He had been very interested in that ever since he had had a discussion with the Bridge Engineer in Amsterdam who, to get very shallow bridges, always employed steel joists in concrete, and said he had never yet succeeded in producing a water-proof deck. The Dutch were very thorough engineers, and no known methods would have been neglected, but this Dutch engineer had said that in every case the trouble was caused by corrosion of the joists in the concrete, and water invariably came through in a very short time, despite waterproof membranes on top. It was necessary, therefore, to accept the fact that the water would get through the waterproof covering; but the Author had been observing the Rhinefield Bridge, which admittedly had only been up for about a year. The surfacing was quite an open textured tarmac with no waterproof value at all, but no water had yet penetrated through the deck. That was what would be expected, because the joints between the adjacent beams were subject to a compressive stress of the order of 75 lb. per square inch—enough to keep the joint tight. That would seem to be the best way of producing a tight deck, and was another point in favour of transverse pre-stressing.

The Author had investigated composite construction. He had been very keen to do the particular job in question in the way in which he did do it, but prudence had dictated that he should obtain alternative quotations from one of the biggest manufacturers of the inverted T-beam. He had found that the costs of those beams was half as much again as the cost of the pre-stressed solid rectangular beam, and after that, of course, the beam had to be concreted up, and possibly temporary scaffolding put under it while it was being concreted. He estimated that the cost of that concreting and of the high-quality topping required would be at least as much as, and probably more than, the cost of transverse pre-stressing, so that the cost of one of those beams would be about one-third as much again. He understood that the manufacturers concerned had recently reduced their prices, but, though that reduction had been appreciable, it was still not sufficient to make the method more economical.

For spans of 50 feet the solid slab would not be economical, as stated in the Paper, and possibly a beam with a cast-in-situ deck might be better. He had not built such a bridge. On spans of that kind, the hollow-box type of construction was probably cheaper than solid slab, and possibly cheaper than T-beams.

The design of the footpath for accidental loading had been taken into account, and a 4-ton accidental load provided for, although the parapet girder design and footpath design were not particularly convenient from the structural point of view. It would be impossible to widen a bridge of that type of construction, and it would be better to arrange the footpath in such a way that it was independent of the bridge, to allow for future widening. From the structural point of view, however, the rather light-looking parapet beam was adequate.

The 2-inch concrete cover would not help very much from the waterproofing point of view. Its weight might have some damping effect (see previous remarks on that point), but a 2-inch screed was bound to crack by the very nature of its substance.

Friction in the cable in the pre-cast capping beam had been considerable ; probably because the plain wrapping of the cable in paper was not enough ; similar curvatures in normal ducts had been stressed without much trouble. It had been necessary in the case in question to stress from both ends. Any cable with a marked curvature should be stressed from both ends ; otherwise the indicated design extension might not reveal that most of the stress was at one end of the beam.

There had been some criticism of the thinness of the lower flange of Test Bridge, Stockbridge, with all the Freyssinet cables in it. It would, of course, be necessary to cast that unit upside down, and the Author was satisfied that, given reasonable workmanship, it could be cast satisfactorily and would present a reasonable appearance on the top surface. The bridge for which it had been designed was one which could not without great difficulty be seen from underneath. The diaphragms were not very finnicky, and they were cast in with the unit. There were in that bridge several hundred units so that mould costs would in any case be low, and the cost of the small amount of additional formwork required for the diaphragms would not be very great, taken over the whole of the units, and would certainly be much less than the cost of casting in situ.

Correspondence on the foregoing Paper is now closed and no further contributions may be accepted.—Sec. I.C.E.

CORRESPONDENCE

On Papers published in Proceedings Part II, February 1952

Airport Paper No. 18

" Gravel Compaction and Testing, and Concrete Mix Design at London Airport " †

by

Harold Smith, A.M.I.C.E.

Correspondence

Dr B. H. Knight asked what number of test holes per 20,000 square yards of compacted gravel sub-base per day the Author would suggest. He agreed that 200 such holes per day would require a considerable staff, but would like to know whether the Author had any reliable data connecting the number of test holes necessary with the probable variations in the in-situ grading analyses of the subgrade gravel used at London Airport.

He was unable to follow the description given by the Author as to the exact method finally used to compact the gravel sub-base. What thickness of layer was used ? Was the sheepsfoot roller used at all and how many passes of whatever roller was used were found to be necessary ?

Finally, he would like to record his view that although any results obtained by the Author might be valid if applied to the predominantly flint gravels of the London Basin, it had not been shown that such results were of general applicability in other areas. Gravels varied widely in their mineralogical composition ; some contained fragments of limestone, dolomite, or igneous rock, and surface texture and particle shape exerted a profound influence on the physical properties of concretes made from them. He assumed therefore that the Author would not claim that his results would hold in the case of gravels of a type other than those dealt with in the Paper.

The Author, in reply, stated that the first point raised by Dr Knight had been answered on p. 7 where it was stated that twenty density tests were taken in an area 600 feet by 150 feet (10,000 square yards). That had been found to be sufficient for the control of the general fill.

For the final layer between the concreting forms, 20 feet apart and usually of about 2 inches thickness, tests had been taken every 100 feet, or less if the area was suspect, or roughly twice the number as for the general fill.

---

† Proc. Instn Civ. Engrs, Part II, vol. 1, p. 1 (Feb. 1952).

It should be noted that those figures could not be used for cohesive soils which were more difficult to compact to a uniform density.

The best method of compacting the gravel was found to be the rubber-tired roller in layers not exceeding 8 inches thick. The number of passes required was related to the nature and moisture content of the gravel. A well balanced gravel with about 30 per cent sand and a ratio of 15 per cent silt to sand and at optimum moisture required only two passes of the roller—one at right angles to the other. That had been done repeatedly at London Airport with perfectly satisfactory results, giving dry densities of 135 to 137 lb. per cubic foot.

With regard to Dr Knight's final remarks, the Author had not intended to show in the Paper details of any other types of aggregates, since the Paper was solely concerned with London Airport.

Dr Knight could rest assured that the Author was well aware of the various types of aggregates used in Great Britain for concrete making and exactly the same principles applied in the design of a concrete mix.

In the equation given by the Author the unknown variables were percentage moisture, specific gravity, and workability, and by substituting known figures for those variables the equation could be easily solved.

The Author had shown that shape had no effect upon moisture content and the curve in *Fig. 5* showed how to determine the minimum required. Specific gravity was easily found, whilst workability was determined by the grading and shape of the aggregate.

The Author had collected, over a number of years, workability figures for various aggregates but felt that it was wrong to publish them; the tendency would be for them to be copied whereas the intention of the Paper had been to show a principle and encourage Engineers to find the workability figures for themselves based on the figures given in the Paper for irregular aggregates.

The question of deleterious matter in the aggregates had not been considered because it had been assumed that allowance would be made or the aggregate rejected entirely.

If surface texture affected the moisture content, as given in *Fig. 5*, then it was quite a simple matter to find the actual moisture by experiment as explained in the Paper. Again, it was a matter of principle rather than one of figures and once the variables had been established for the aggregate to be used then the design of a concrete mix was simplified.

"Economy in Railway Civil Engineering" †

Correspondence

Mr J. T. Cosgrove remarked that Mr Stevens had stated that the lowest productivity observed under day-work had been more than 30 A.Ms per hour. In other Regions where the Depots were smaller, and usually run by an inspector, where a certain method and output had been accepted for years, and where costing was not done locally, productivity in certain sections may well be found to be in the 'teens, and Mr Cosgrove thought that 30 A.Ms per hour would represent a fair average for that type of Depot.

The task of bringing productivity up to the level that would make an incentive scheme attractive to the men was therefore of considerable magnitude, though the possible economies to management would make it correspondingly profitable.

Mr Terris got to the root of the problem in his notes on supervision. One point he stressed was the need for personal contact. It was generally recognized in the Army that the Officer, non-commissioned or otherwise, who obtained the best results was usually one who exerted his personality (for the good) and ensured that his men were thoroughly "in the picture." If this was found to be desirable within the strict discipline of the Army Act, how much more desirable was it in industry, under the present conditions of employment.

Mr Cosgrove referred to a small Depot where useful productivity had risen approximately 125 per cent, without any direct financial incentive or change in method, as a result of the men being impressed with the need for maximum production at an economic price. Productivity had not been exceptionally low beforehand, nor had the men's effort increased by quite that extent, but a moderate increase in effort had enabled the management to eliminate double handling and other non-productive work, and so obtain greater production and lower costs.

Other opportunities might occur to break down restrictive practices by the introduction of new methods, such as were associated with 109-pound flat-bottomed rail; thus a "new tradition" could be established.

If performance be increased by any such means, then the ground would be prepared for an incentive scheme which might well be based on a starting level of 60 A.Ms per hour, thus giving double the financial incentive of a scheme based on 40 A.Ms per hour.

Outside the confines of Production Depots, there were tremendous possibilities in linking the estimated man-hours, which Mr Taylor Thompson described under the Cabinet system of Works Control, with A.M. values.

---

† Proc. Instn Civ. Engrs, Part II, vol. 1, p. 97 (Feb. 1952).

Mr Stevens had mentioned, in his oral introduction, that the additional administrative staff required amounted to 2 per cent of the number of men employed. Even if such an economical percentage were not attainable in smaller Depots, it would be very false economy to dispense with that administrative staff in any future emergency, even if the scheme were running smoothly. Without constant attention, any incentive scheme would get rapidly out of hand.

The Author thanked Mr Cosgrove for his contribution but did not feel called upon to make any comment.

CORRIGENDA

Proceedings Part II, February 1952

Airport Paper 18 (Smith)

p. 41, para. 3, line 8.—*For* " 0·45 " *read* " 0·55 "

Paper 5833 (Swansbourne)

p. 210, footnote.—*For* " 1st October " *read* " 1st August."

Paper 5845 (McIntosh and Mercer)

p. 220, Synopsis, line 2.—*For* " 1949 " *read* " 1950."
p. 220, Introduction, line 1.—*For* " 1949 " *read* " 1950."

ADVERTISEMENT

The Institution of Civil Engineers as a body is not responsible either for the statements made or for the opinions expressed in the foregoing pages.

MADE AND PRINTED IN GREAT BRITAIN BY WILLIAM CLOWES AND SONS, LIMITED
LONDON AND BECCLES.